Concrete Bridge Strengthening and Repair

Iain L Kennedy Reid

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Cover photograph: Top of crossbeam and panel wall water jetted out for repair

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Preface

The following case studies are summaries of work undertaken by Atkins Highways & Transportation during their maintenance of the Midland Links Motorway Viaducts for the Highways Agency over the past nineteen years.

The document forms a sister to Atkins Report entitled *Steel Bridge Strengthening: A Study of Assessment and Strengthening Experience and Identification of Solutions* carried out as a project for the Highways Agency and subsequently published by Thomas Telford.

Iain L Kennedy Reid
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Acknowledgements

Atkins are grateful to the Highways Agency for their kind permission to publish this book.

This book is dedicated to the memory of my parents, to my partner and her family, and to the many Atkins' staff who planned, designed and enabled the remedial works to so many different concrete elements.

Iain L Kennedy Reid
Design Manager
Atkins Highways & Transportation

Introduction

Concrete Bridge Strengthening and Repair seeks to make available to the reader the benefit of experience gained by Atkins in strengthening and repairing a wide range of concrete bridge elements resulting from a number of different types of defect.

Twenty case studies are presented illustrating various types of damage and remedial techniques to beams, slabs, columns and panel walls. Case 1 is a full-blown study of the diagnosis of apparent shear cracking in a reinforced concrete motorway crossbeam subject also to chloride attack, corrosion and delamination. Alternative strengthening and repair options are described, as is the approach to managing the risk of failure, including load restrictions and emergency propping. The type of steel propping devised was compatible with that already being developed for the repair of other crossbeams within the viaducts, and comprised a 'Meccano-style' concept of steel units which could be erected in different formats to suit the geometrical variations between adjacent crossbeams. The strengthening method adopted is also described in detail.

Case 2 describes the design and construction of the temporary and permanent works involved in replacing a full-width motorway reinforced concrete crossbeam under live traffic. Design options considered are summarised, and the changes in articulation of the motorway deck during the works are described in detail, together with the theoretical, practical and dynamic aspects of the measures developed to accommodate these articulation changes. The jacking procedures adopted to transfer the deck load on to temporary supports during the replacement of the crossbeam are also treated in some detail.

The repair and strengthening of panel walls is described in case 3, covering shear cracking due to deck shrinkage, chloride attack and misplaced reinforcement. Concrete removal and reinforcing bar repair are both described in some detail. Case 4 describes the repair of concrete crossbeams, extending the description of rebar repairs, covering the degree to which smaller repairs could be carried out unpropped, addressing the theoretical approach to repair under propping, and describing the sequence of repairs.

In case 5, the repair of cantilevers is addressed, introducing the use of one-third scale model testing to determine the deteriorated and repaired strength of reinforced concrete beams, and linking the results to the management of the actual structures in terms of applying load restrictions pending repair. Bond testing is also mentioned, as is the use of Professor Regan's method of assessing the combined effects of shear and moment on the capacity of reinforced concrete beams. Having no redundancy, cantilevers are particularly vulnerable and the method adopted of taking account of the weakness of existing delamination adjacent to areas under repair is demonstrated. In case 6 the particular aspects of repair below bearings are described, showing how deck beams were jacked up and stabilised to permit the repairs, together with the special treatment required at the deck edge beams.

Very large voids were found in some crossbeams when they were exposed for repair and the discovery, investigation, causes, management of safety of the viaduct and interim and long-term proposals for treatment are described in case 7. Columns are treated in cases 8, 9, 11 and 14. These cover respectively, repairs to built-in and hinged columns and deterioration and replacement of columns. Contamination, emergency and repair propping, steel strapping, repair sequence and reinforcement repairs are all described in some detail. The use of models to assess the adequacy of access for repair is demonstrated, together with some difficulty encountered during dejacking of a replaced column.

Further studies of crossbeams are included in cases 10, 12 and 13. In case 10 the condition of a severely deteriorated crossbeam is described resulting from a significant shortfall in cement content when originally cast, exacerbated by chloride attack. In cases 12 and 13 sagging and hogging weaknesses found during assessment are detailed, together with descriptions of the resultant cracking and options for strengthening by the insertion of an additional column and the casting of reinforced concrete nibs on the beam corners.

In case 15 an interesting procedure of removing a concrete deck downstand and replacing it with discrete bearings to permit access to the contaminated crossbeam below for the application of cathodic protection, is described. An adjacent procedure is also addressed for the replacement of corroded circular stub columns, supporting a skewed deck end, with a steel trimmer beam.

Decks are further treated in cases 16, 18 and 19. Repairs to corroded reinforcement and contaminated and delaminated deck end concrete are addressed in case 16, together with the development of provisional repair sequences prior to deck resurfacing. In case 18 the non-linear analysis of a length of viaduct adjacent to an opening deck joint is described, which demonstrated that the viaduct was not overstressed and that the replacement of the expansion joint with one of greater movement capacity would solve the problem. In case 19 a situation is discussed by which high bearing friction values, in requiring the regrouting of bearing plinths, resulted in a deck beam end being accidentally dropped during the jacking process, leading to delamination and subsequent corrosion and spalling of the adjacent area of deck soffit. Proposals were made for grouting and spray concreting of the deck soffit, although in the event the deck was repaired from above during resurfacing.

Column strengthening against vehicle impact by the insertion of concrete slab panels between rectangular columns is described in case 17 while a most interesting study of a combination of shrinkage, contraction and ASR is discussed in case 20. In the latter case, concrete crossbeams were subjected to high transverse forces resulting from excessive bearing friction. This, combined with shrinkage, led to longitudinal cracking which in turn provided access for additional moisture to generate ASR. The programme of investigation and monitoring is described, together with traffic management, interim strengthening, and long-term repairs.

Case 1 Crossbeam strengthening

Summary

This case study describes the condition of a Midland Links Crossbeam which exhibited diagonal cracking and was also in an ongoing state of corrosion due to chloride contamination from road salts. The approach to diagnostic analysis is discussed which eventually determined the cause of the diagonal cracking. The case study highlights the fact that settlement of foundations can in some circumstances seriously affect the results of structural assessment.

Details are given of the method of temporary propping pending strengthening and repairs which comprised a universal plate girder supported on the existing foundations together with a counterweight to avoid uplift.

The means of repair of the corrosion in the reinforcement and of the contamination in the concrete is described, together with a possibly novel solution for strengthening the crossbeam to avoid further diagonal cracking.

The propping, strengthening and repair works were all completed successfully on site; the universal girder concept was used extensively for the Midland Links repairs; and once the propping was removed the diagonal cracks did not re-open and there was no evidence of further diagonal cracking.

The problem

The Midland Links Motorway Viaducts carry the elevated section of the M5 and M6 Motorways on the outskirts of Birmingham in the Midlands area of England. The viaducts are mainly constructed of standard sections of steel concrete composite decks simply supported on reinforced concrete crossbeams and circular columns. Build around 1970 the viaduct joints leaked, allowing de-icing road salts to contaminate the crossbeams with chlorides, resulting in reinforcement corrosion and delamination of the concrete cover. The crossbeams are now undergoing an extensive programme of repair of the delaminated areas and the application of cathodic protection.

One of the largest crossbeams 30 m in length, 2 m deep and 1.7 m wide, traversing a canal and supported on three 1.5 m diameter columns, exhibited diagonal cracks some 15 years earlier. These were monitored and found not to be moving. However, some 10 years later additional diagonal cracks appeared and the degree of corrosion and delamination of the crossbeam was steadily worsening (see Figure 1.1).

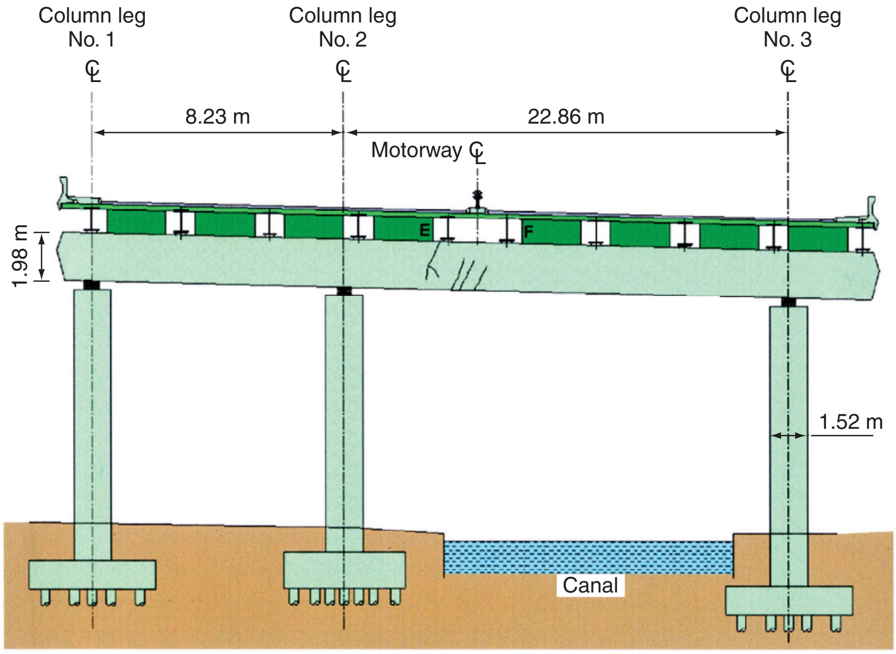

Figure 1.1 – Crossbeam elevation showing cracks

Diagnosis

The crossbeam was therefore assessed in its deteriorated condition. The actual loss of section of rebars was not considered to be significant and the crossbeam was found to have adequate shear strength provided by numerous links. The force in the longitudinal reinforcement was determined by the rigorous method proposed by Regan[1] allowing for the combined effects of shear and bending. The adequacy of the delaminated bond strength to provide sufficient anchorage at the point of curtailment at the location of the diagonal cracking was assessed on the basis of earlier model testing[2] and found to be just adequate. There appeared, therefore, to be no good reason for the diagonal cracking, particularly as settlement of the central column would reduce the applied shear at the crack location.

Nevertheless the structure was checked for signs of settlement. The bearing on the central column was found to be 15 mm lower than those on the outer two columns. To check whether this was due to settlement or to construction, the soffits of the deck beams were levelled and plotted and then corrected for long-term dead load deflection. The result was a settlement of the middle column by about 17 mm. The piled foundations were back analysed for anticipated settlement, and, due to its large size and applied loading, the middle foundation was found to have been susceptible to a settlement of, amazingly, 17 mm!

This settlement was then applied to the assessment of the deteriorated crossbeam and found, due to the addition of sagging moment, to increase significantly the force in the longitudinal reinforcement (see Figure 1.2). The capacity ratio dropped to 0.65 under 45 units of HB loading. A load restriction of 100 tons was immediately imposed on the structure and the hard shoulder closed.

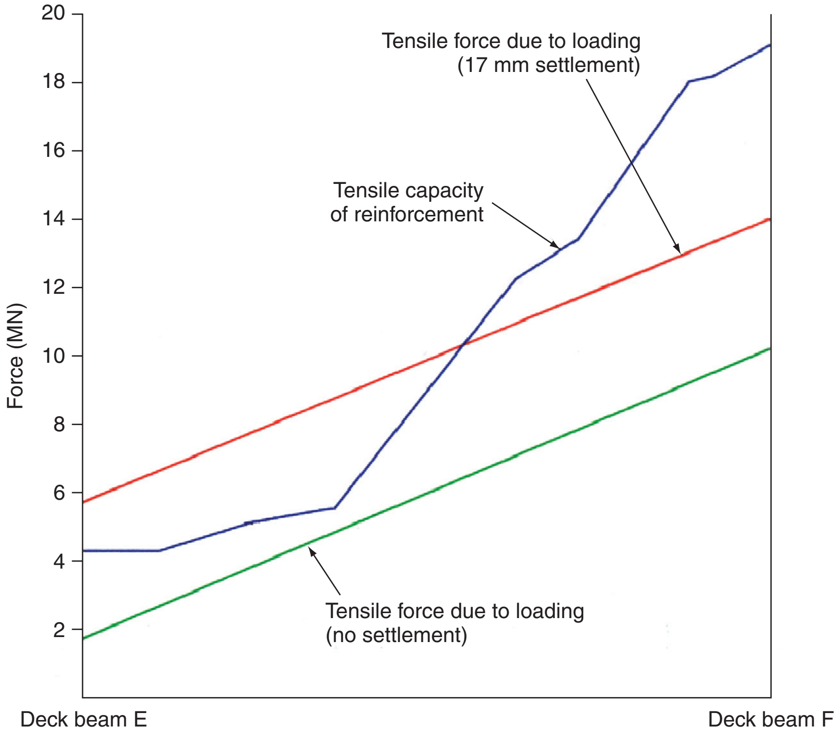

Figure 1.2 – Capacity of main bottom reinforcement

The diagonal cracking was then re-examined and found to link zones of rapid curtailment of top and bottom longitudinal reinforcement which was in effect badly detailed (see Figure 1.3). This combined with the overstress of the longitudinal reinforcement due to the settlement in the zone of high shear had led to the diagonal cracking.

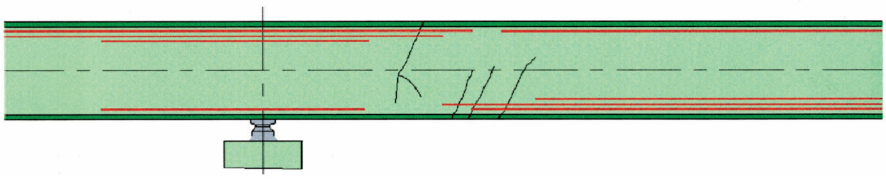

Figure 1.3 – Beam elevation showing main reinforcement and cracks

Strengthening solution

Means then had to be devised as to how the crossbeam could be strengthened. External prestressing was considered, but the beams have little vertical and no longitudinal reinforcement in the side face, so anchorage to the sides of the beam would have been a problem. Fixity to the top or bottom of the beam would also have been difficult as the longitudinal bars are so closely spaced that bolting between them, and missing the links, would have been problematic (see Figure 1.4).

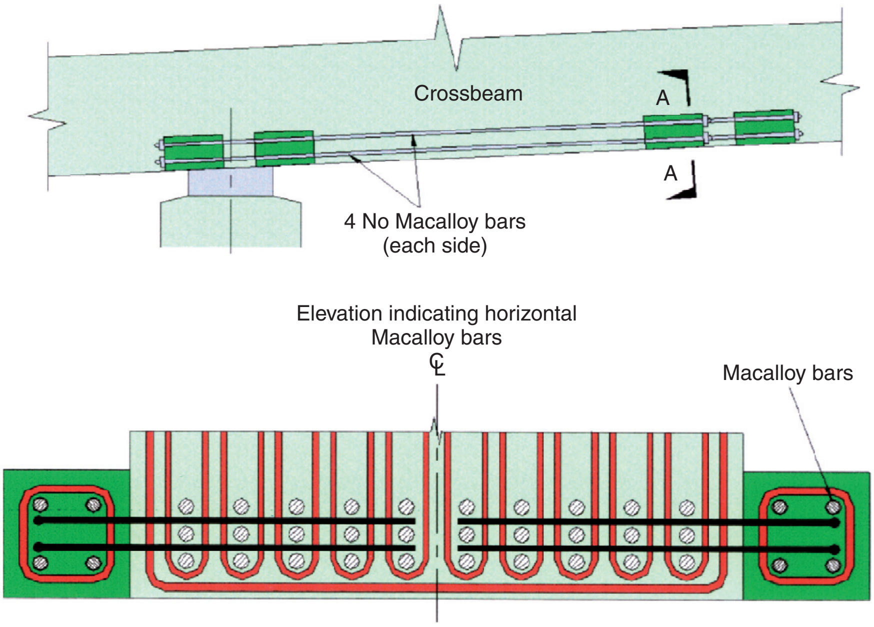

Figure 1.4 – Macalloy bar proposal

Augmenting the reinforcement on the top and bottom of the beam was then considered but the panel walls fixed to the top of the beam and the jacks supporting the beam from temporary propping below would interfere with fixing the rebars and again the close spacing of the existing reinforcement discouraged the installation of shear connectors.

Use was then made of the fact that much of the top and bottom of the beam would be systematically removed for repair of delamination. Reinforcement could be cast into longitudinal nibs provided at the beam corners and stitched into the top and bottom of the beam during repair. This would solve the problem of fixity to the beam and avoid conflict with items at the beam top and bottom. In the event this solution proved to be very successful and no major problems were encountered during construction (see Figure 1.5–1.7).

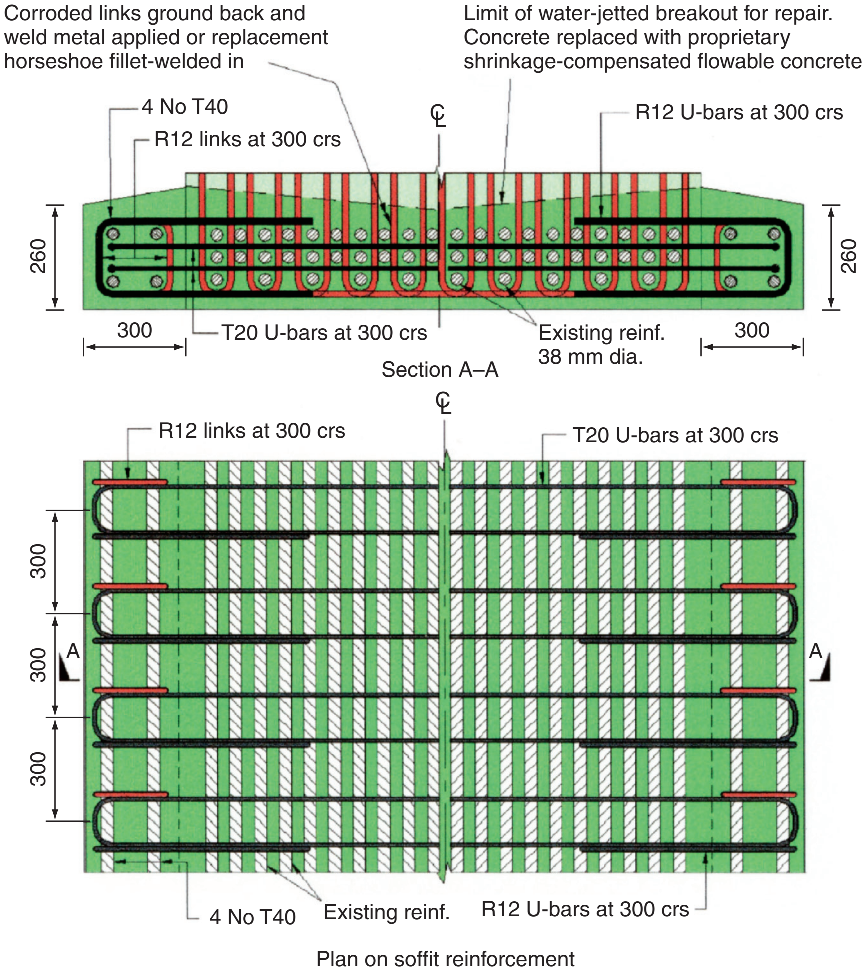

Figure 1.5 – Nib reinforcement

Figure 1.6 – Top of crossbeam broken out to enable insertion of anchor bars for strengthening nibs

Figure 1.7 – Short length of nib cast

Propping

Once the capacity ratio was calculated during the assessment allowing for the settlement, the decision was made to prop the crossbeam as soon as possible. Meantime the crack widths were monitored. Standard crossbeams on the Midland Links were propped for repair with standard trusses and plate girders supported on trestles erected around the columns, such that the temporary steelwork could carry all the dead and live loading. This crossbeam however was one of many on the Midland Links crossing hazards such as canals which required the columns to be at non-standard spacing. A ‘meccano-style’ system was therefore devised of plate girders which could be braced together in a variety of ways to avoid columns at different locations. The crossbeams supported by three columns were grouped together so that the longest crossbeams carrying the longest deck spans were covered by the largest plate girder the Y1, the next largest covered by the Y2 and so on to the Y4. The crossbeams supported by two columns were covered by the X1 girder, and those crossbeams with four or more columns were covered by the Z series.

As the columns were at imperial spacings, stiffeners were welded at a 2 foot modular spacing, and heavier bearing stiffeners were fixed to cover all bearing locations for each potential use of a particular girder. Plan bracing for all the girders could be fixed at 6 foot, 8 foot or 10 foot spacings and the permutations involved could cater for any column location. Standard holes were drilled in the flanges and stiffeners of each beam on the 2 foot module so that the bracing could be fixed at any 2 foot location. The holes in the 8 foot bracing members were straight, while those on the 6 foot and 10 foot members were skewed one way or the other so that any of the three plan bracing members could be bolted to the standard holes in the flanges (see Figure 1.8). The crossbeam described in this case study was propped by the Y1 girder (see Figure 1.9).

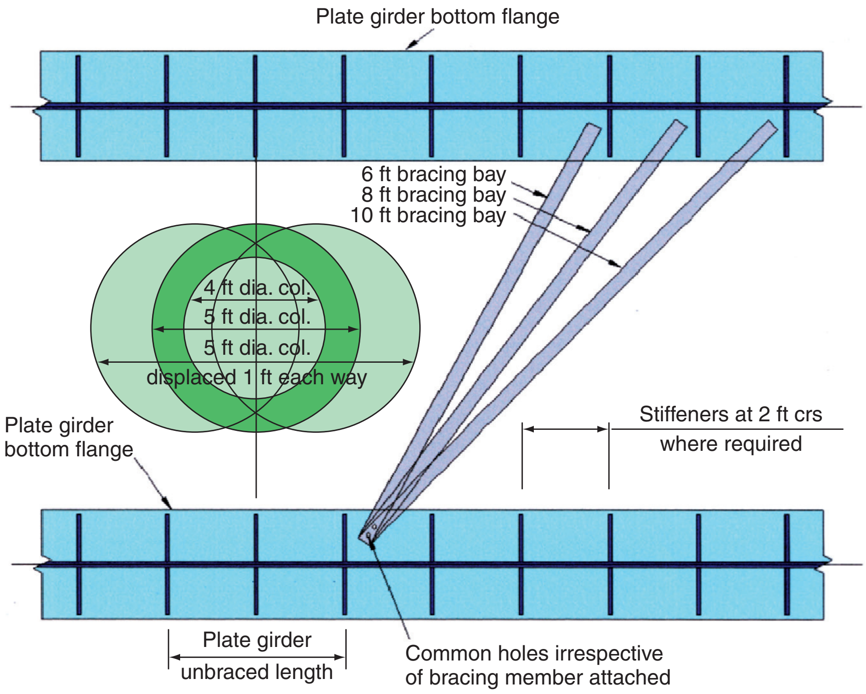

Figure 1.8 – Universal propping

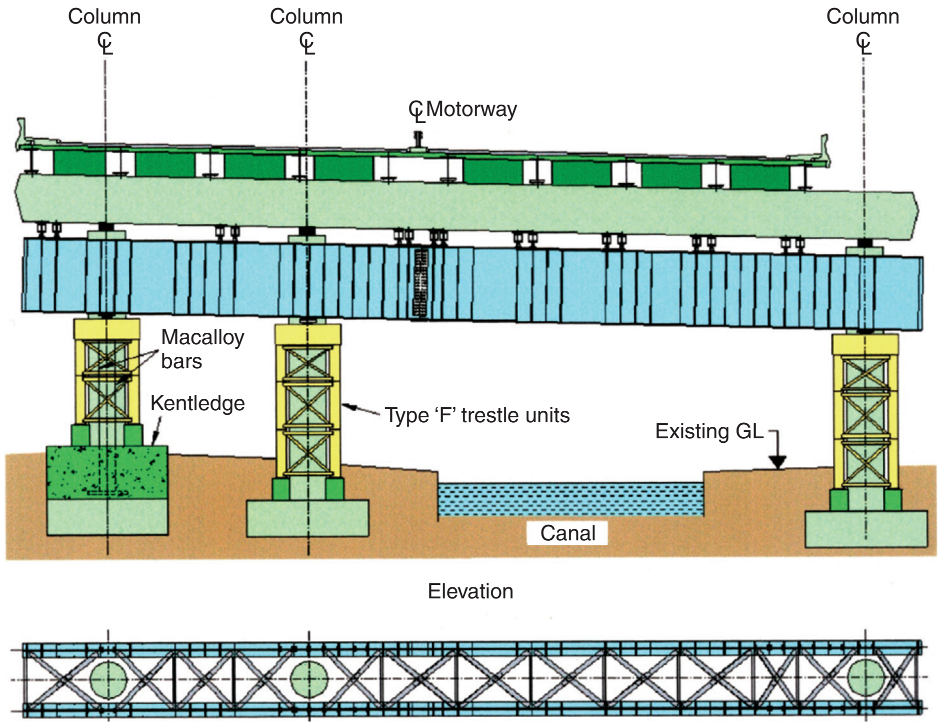

Figure 1.9 – Crossbeam supported by universal plate girder

Once propped the dead load was jacked out of the crossbeam into the plate girder so that the crossbeam could safely be broken out at top or bottom over half its width for repair. The stiffness of the crossbeam relative to the temporary steelwork meant that only 20–30% of the live load was attracted to the plate girder (see Figure 1.10).

Figure 1.10 – Plate girder erected across canal

Kentledge

Being supported by three columns, the bay length of the crossbeam between the two columns spanning the canal was much longer than the bay length between the other two columns. Steel rocker bearings supported the crossbeam on the permanent columns and to avoid lifting off these bearings the dead load could not be jacked out of the crossbeam immediately over the columns. The result of these two factors meant that under certain live load conditions the plate girder could lift off its bearing at the outer end of the short bay. This was avoided by casting concrete kentledge on the existing foundation and tying the plate girder down to the kentledge by Macalloy bars. The Macalloys could not be fixed to the foundation because there was no top reinforcement in the foundation.

The kentledge would result in a certain calculated settlement of the foundation but this would reduce the relative settlement of the central pier and so relieve the overstress on the crossbeam.

Tying down the plate girder on the end support ensured that the jacked crossbeam would not lift off its own column bearing (see Figure 1.11).

Figure 1.11 – Plate girder tied down to counterweight

The plate girder was successfully erected over the canal and the crossbeam propped. A separate contract was then let for the strengthening and repair of the crossbeam.

References

1. Regan, P. E. (1985) *Shear Concrete Society Current Practice Sheet No 105,* Concrete November 1985.

2. Department of Transport Midland Links Motorway Viaducts, *Bond Tests Working Paper No WSA/1/88,* WS Atkins.

Case 2 Crossbeam replacement

Summary

The deterioration of bridge substructures caused by corrosion induced by de-icing salts, is a common problem for the UK's bridge stock. Generally piecemeal repairs are carried out but complete replacement is preferable in terms of the durability of the final product. This case study describes the development of a scheme for replacing a motorway crossbeam support from concept to successful completion. The problems encountered especially with respect to undertaking the work with minimum traffic disruption, are described in detail. Similar projects are reviewed and recommendations made for further developments.

The problem

The Midland Links Motorway Viaducts carry the M5 and M6 motorways around the suburbs of Birmingham, UK and comprise over 1200 spans of elevated structures. These are generally simply supported steel and concrete composite bridge decks supported by reinforced concrete crossbeams and columns.

De-icing salt leaked onto the crossbeam supports through leaking joints, causing such widespread corrosion that the majority of the supports had to be repaired. A major maintenance programme was undertaken with most of the crossbeams being repaired and cathodically protected to prevent further deterioration.

However, some crossbeams were found to be in such a serious condition that they were considered to be almost beyond repair. A scheme was therefore developed to replace crossbeams completely and to carry out this operation with the minimum of disruption to motorway traffic.

The objective

The crossbeam which required replacement carried the dual three lane M5 motorway which has a typical traffic flow in excess of 65 000 vehicles per day. The crossbeam was 33 m long, 1.68 m wide and 1.52 m deep supported by two 1.52 m diameter columns at 18.3 m centres. One column 8.3 m in height is supported by a 6.0 m diameter spread footing founded on stiff clay. The other column which is 5.1 m high is founded on a 6.0 m diameter base supported by 16 No. piles end bearing on dense gravel or stiff clay. The decks are 15 m span, simply supported, each with 10 No. steel universal beams acting compositely with in-situ reinforced concrete slabs. Reinforced concrete panel walls connect the crossbeam directly to the deck slab providing transverse and longitudinal restraint to the superstructure and precluding the need for bearing stiffeners. The bearings were sliding steel on steel with a steel rocker providing longitudinal rotational capacity. The basic configuration of the deck and supports is indicated in Figure 2.1.

The crossbeam was considerably deteriorated due to corrosion of the top and bottom reinforcement caused by a combination of chloride contamination and lack of cover on the soffit (see Figure 2.2). There was approximately 50% delamination of the concrete surface and considerable loss of section of links and main bars. If the beam had been repaired by conventional methods of replacement of contaminated concrete, then 90% of the surface area would have had to be removed. For cathodic protection then all the delamination would have had to be repaired. The main reason for replacement, however, was the condition of the reinforcement and the practicability of repair. The basic requirement was therefore to remove the crossbeam completely and to construct a similar replacement. This work would have to be carried out with the minimum of traffic disruption.

Figure 2.1 – Deteriorated crossbeam soffit

In the case of this particular crossbeam it was located adjacent to a canal on one side and an embankment on the other side which interfered with access and affected the structural solutions (see Figure 2.2).

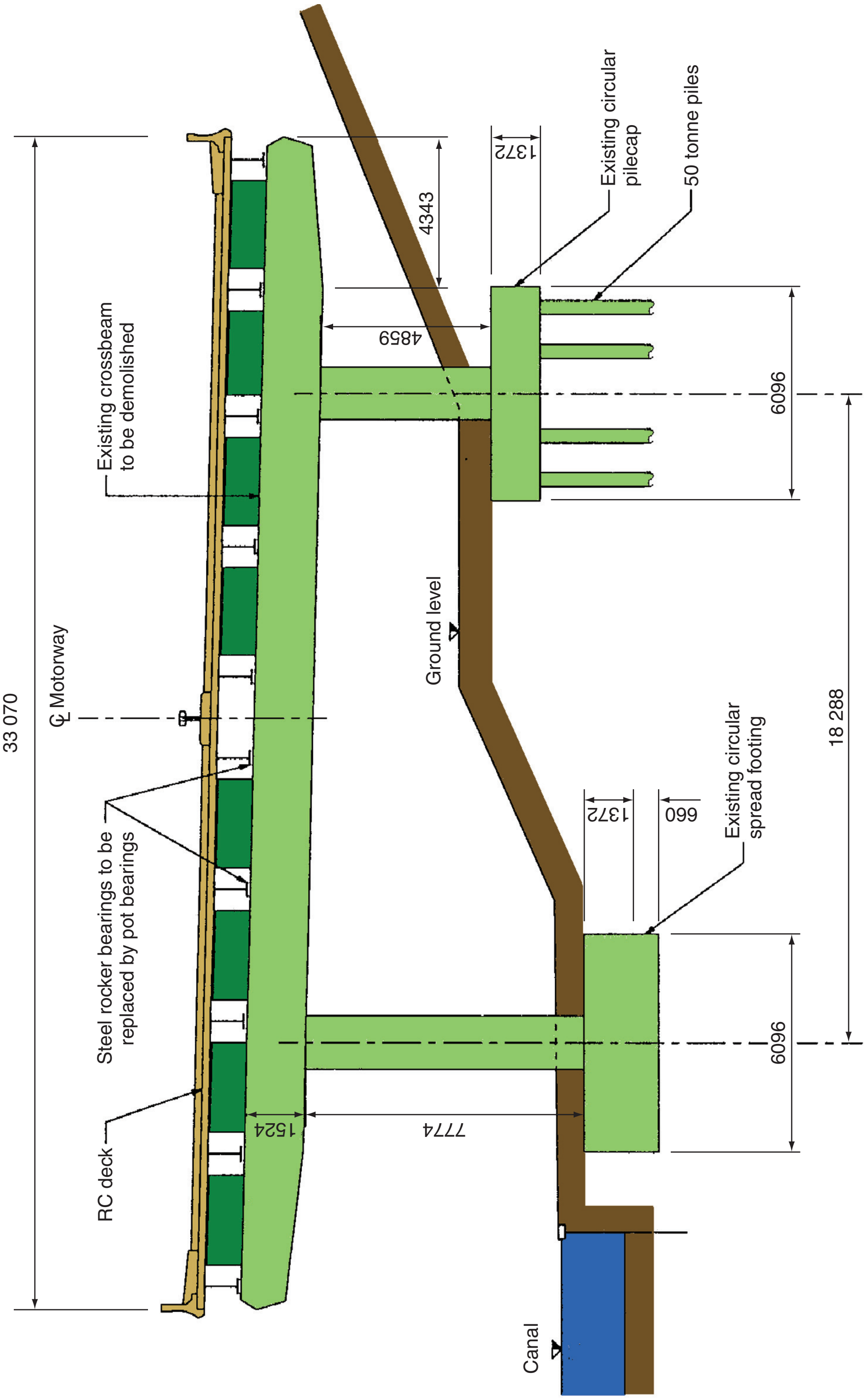

Figure 2.2 – General arrangement of crossbeam

Safety and reliability

An over-riding consideration in the design and execution of this scheme was the safety of the deck structure because of the consequences of affecting traffic on the M5 motorway. When the permanent supports were removed then the temporarily supported structure was much more vulnerable to accidental problems. While catastrophic collapse was the major consideration, with possible loss of life, the economic costs would have been enormous. If the M5 motorway had to be closed, then the resulting traffic delay costs would be in the order of £1 million per weekday. For this reason the scheme had to take account of unforeseen difficulties. Consequently a highly redundant design was preferred in order to increase the reliability of the structure.

The options

A number of schemes were investigated at the feasibility stage. These are described below and are shown in Figure 2.3.

Deck supports

The most obvious scheme involved supporting the decks each side of the crossbeam leaving access for demolition and reconstruction. The temporary supports however required independent foundations.

Beam to one side

This scheme involved constructing a new crossbeam and foundation on one side of the existing crossbeam with a halving joint to support the far span. Although there would have been considerable technical difficulties with the halving joint and strengthening the shear connectors on the existing beam it was an attractive option as it carried less risk. Temporary supports were not required and the new support would not be contaminated by leaking joints in the future.

A variation of this scheme involved constructing temporary supports on one side so giving good access to demolish and reconstruct the crossbeam.

Beam under

This was a modification scheme rather than replacement and while it was attractive in terms of cost it did not achieve the objective of eliminating the present deteriorated crossbeam.

Conclusion

The principle of the first scheme was considered the most suitable and was therefore taken forward to the detailed design stage and is described in more detail in the following sections.

Temporary steelwork supports

The temporary supports were required to cantilever over the canal on one side and oversail an embankment slope on the other.
The scheme devised consisted of a steel frame with inclined legs cantilevering over the canal with a large concrete counterweight (see Figures 2.4 and 2.5).

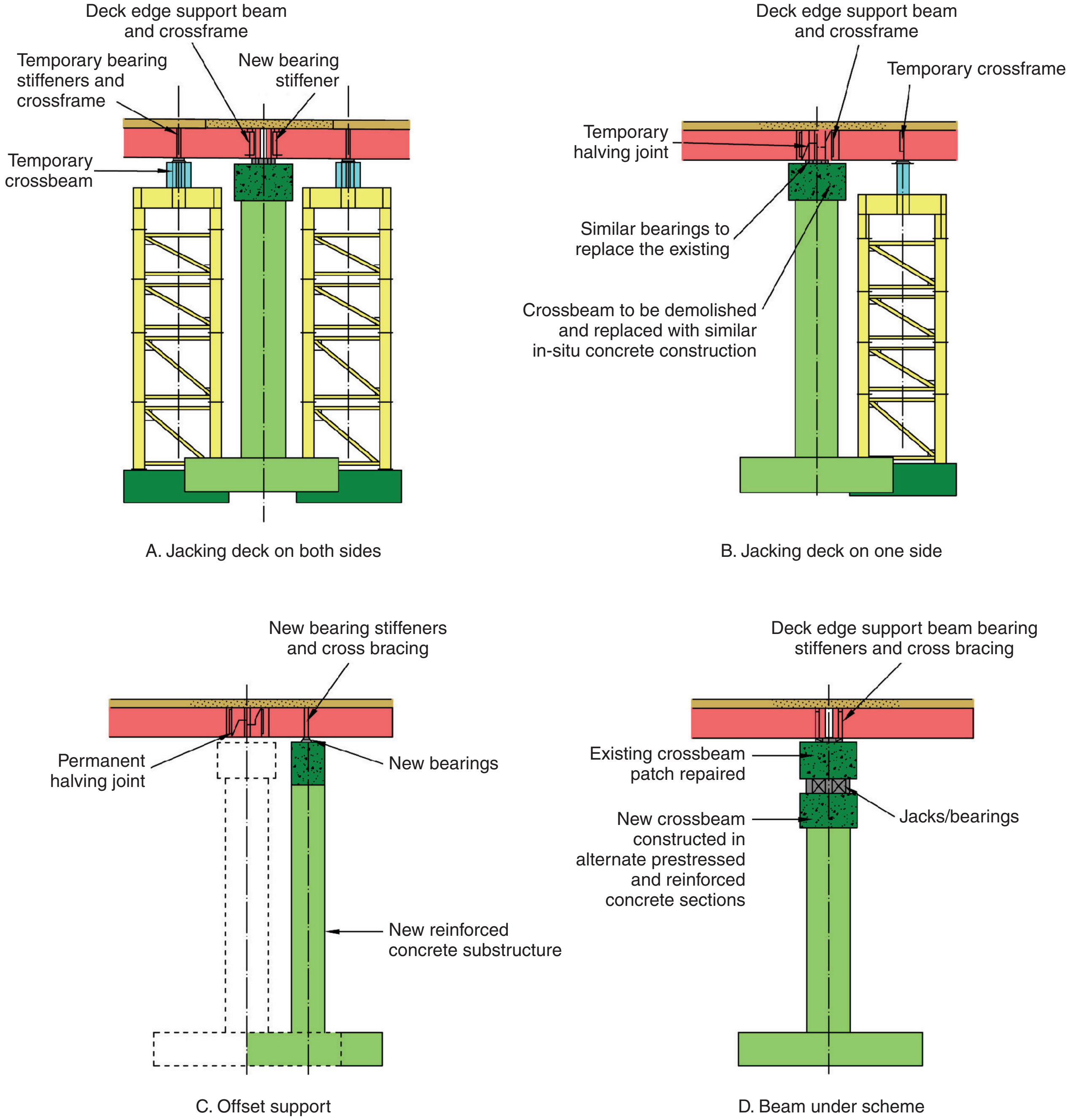

Figure 2.3 – Bridge modification

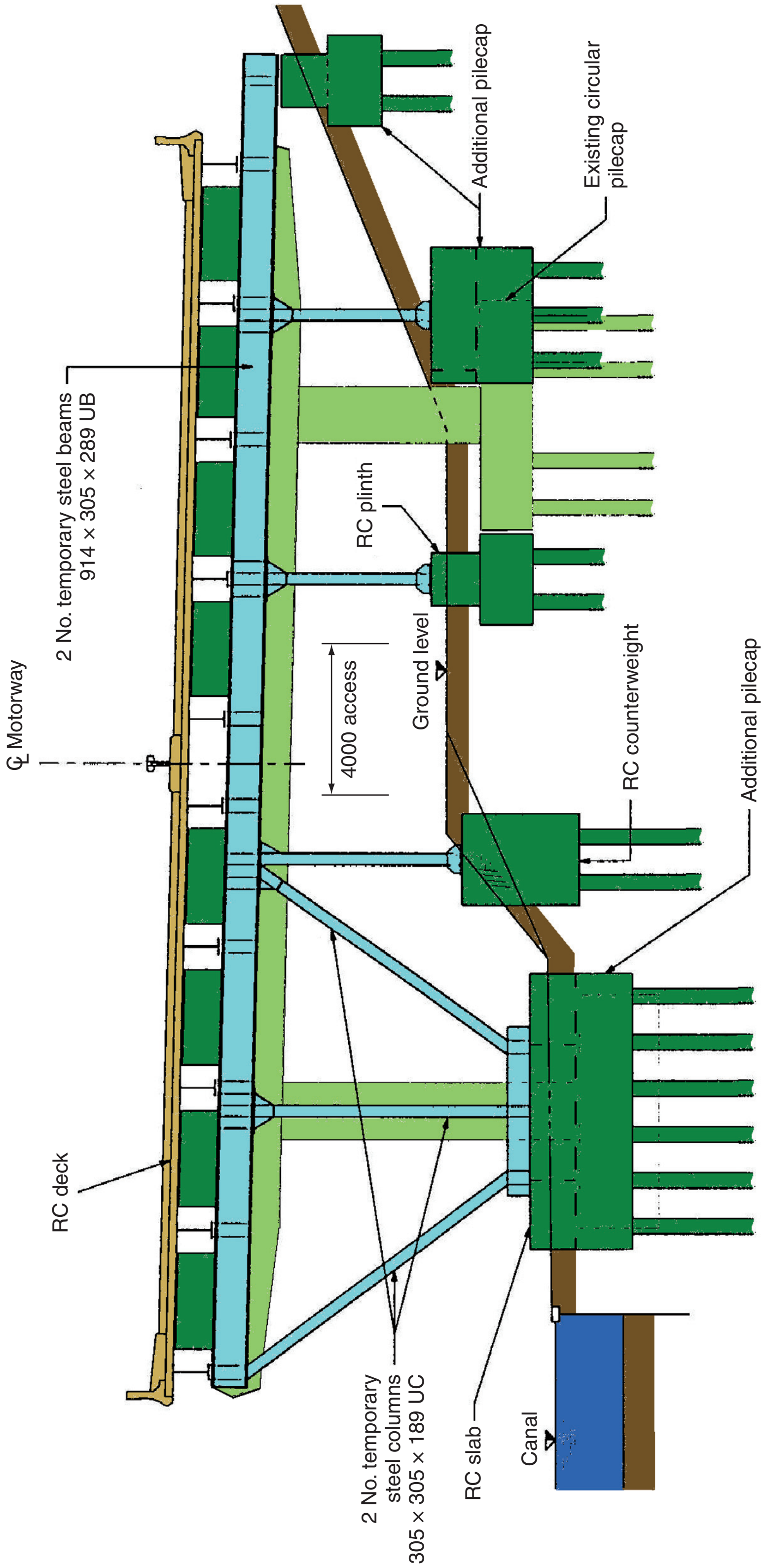

Figure 2.4 – General arrangement of temporary support

Figure 2.5 – Temporary deck supports

Transverse stability of the main beam members was achieved by pairing up the frames and interlinking them with tie beams which provided a suitable seating for both the jacks and the temporary pot bearings. This lent itself to support by twin columns (see Figures 2.7 and 2.8). The frames were also required to deflect with longitudinal temperature movements of the deck as they relied on longitudinal fixity for their stability in that direction.

Deck beam restraint brackets were fastened between the temporary supports and the deck beams to provide longitudinal and transverse fixity during jacking (see Figure 2.6). After jacking the temporary pot bearings provided this fixity.

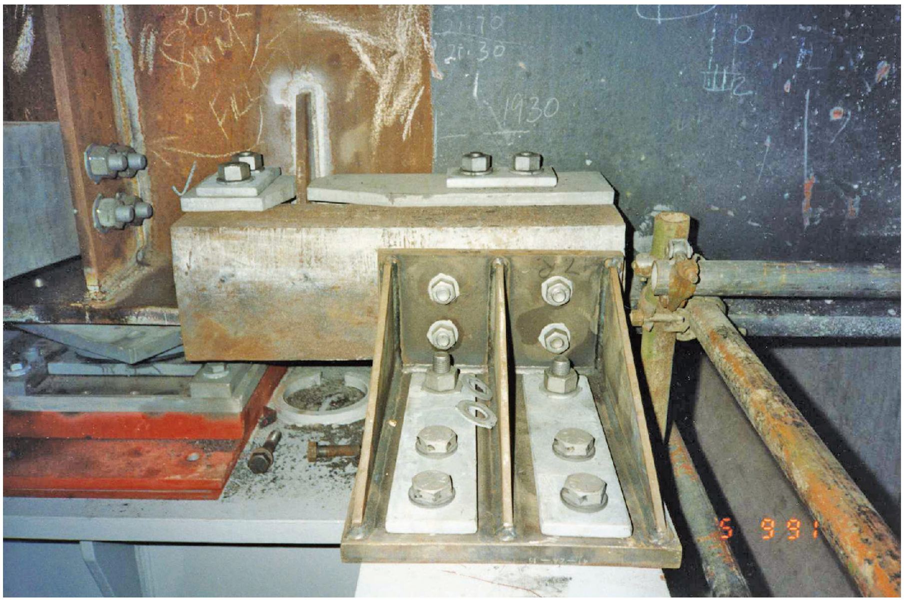

Figure 2.6 – Deck beam restraint brackets

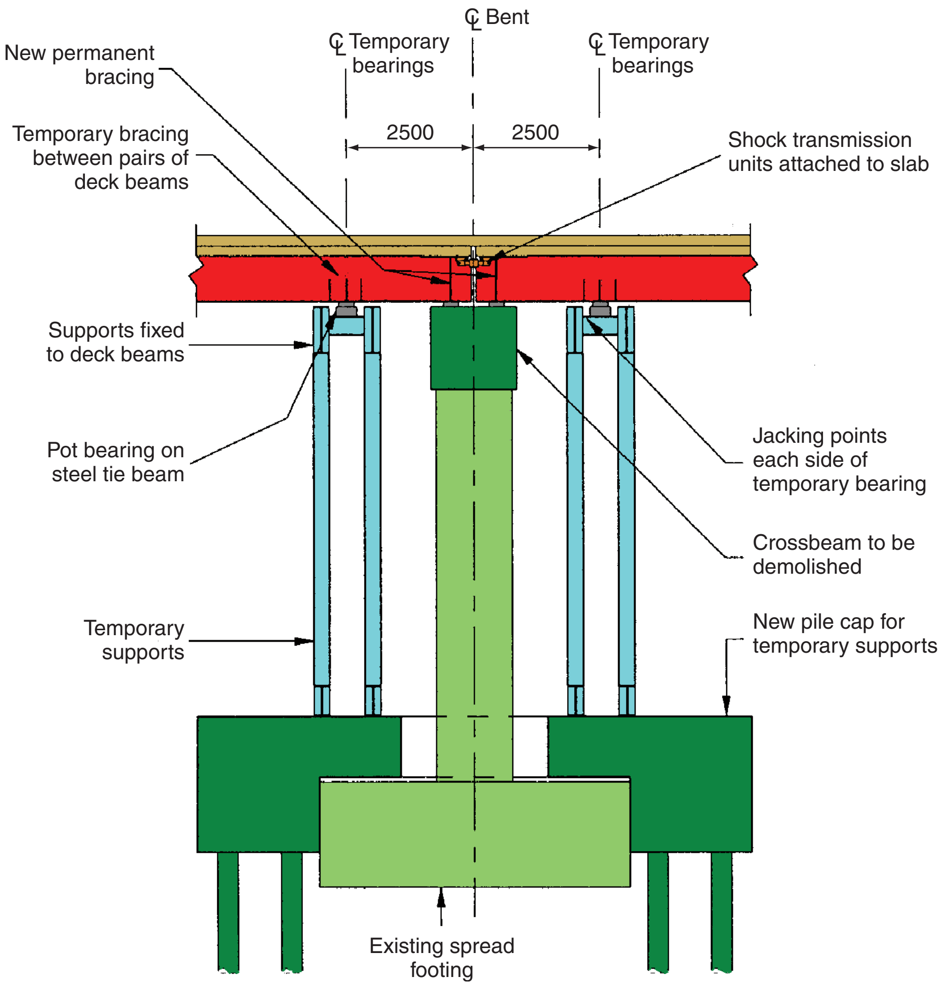

Figure 2.7 – Temporary support details

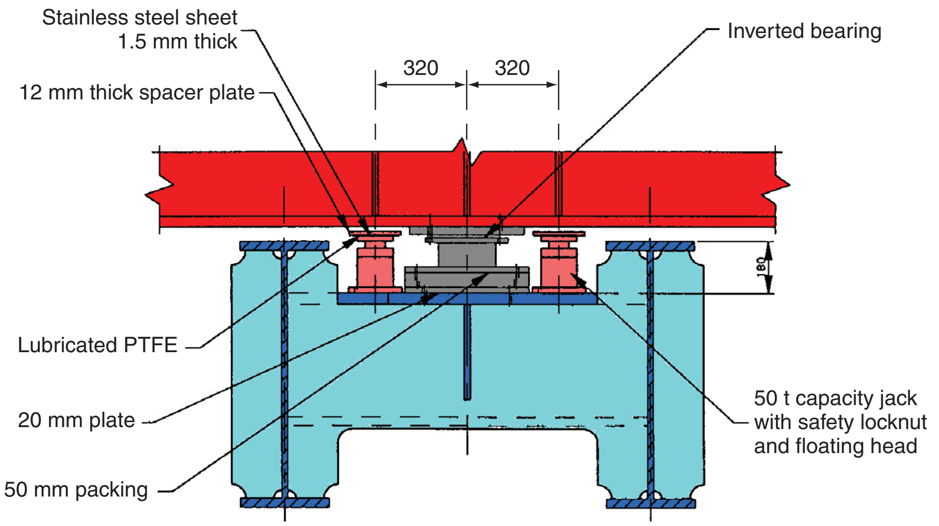

Figure 2.8 – Jacking and temporary beam details

Additional safety was provided by shimming the crossbeam directly under the deck beams enabling the deck and limited live loading to be carried by a single leaf of the frame in the event of damage to the columns of the other leaf. In such an event the deck beam restraint brackets would again provide longitudinal and transverse fixity.

The temporary supports were founded on reinforced concrete slabs bridging between additional piles bored either side of the existing foundations. The bridging slabs were designed to carry possible propping of the existing beam during demolition and falsework for construction of the replacement beam.

Modifications to the existing structure

In order to jack the structure off its existing crossbeam support it was necessary to modify the structure for a number of reasons.

Bracings

Firstly, the panel walls connecting the crossbeam directly to the deck slab had to be removed otherwise the deck could not be separated from the crossbeam. As the panel walls provided transverse stability and longitudinal restraint as well as support to the ends of the decks, they were replaced by K-bracing as is shown in Figure 2.9. Positive support to the ends of the slab deck was provided by pumping grout into special bags placed between the top bracing member and the deck soffit as is shown in Figure 2.10.

Bearings

An additional set of temporary bracing was also required to stabilise the deck at the points of temporary support. This is shown in Figure 2.11.

In the final structure the existing steel on steel bearings were replaced with conventional pot bearings in order to provide transverse rotational capacity at the new bearing stiffener locations.

Shock transmission units

The ‘floating’ articulation was changed to fixed/free and in order to share longitudinal loads between bents, shock transmission units (STUs) were provided. These were also necessary for the temporary propped condition in order to share longitudinal loads between the adjacent bents. See further discussion on viaduct articulation below.

The STU characteristics were specified following calculations of the time for which traction loads would be applied. The movement of the STU had to be sufficiently small during the period of application of the traction load to sustain sufficient resistance between the deck ends. The resistance of the STU to temperature movements, given their extended period of application, had to be sufficiently small not to overstress the viaduct supports.

The STUs were pinned to brackets welded to large plates bolted to the underside of the deck. Adequate tolerance had to be permitted in the bolt location to avoid the reinforcement in the deck. The plates oversailed the trimmer beam of the K-bracing and transmitted shear forces and moment to the deck. They had to be braced to provide sufficient strength (see Figure 2.13).

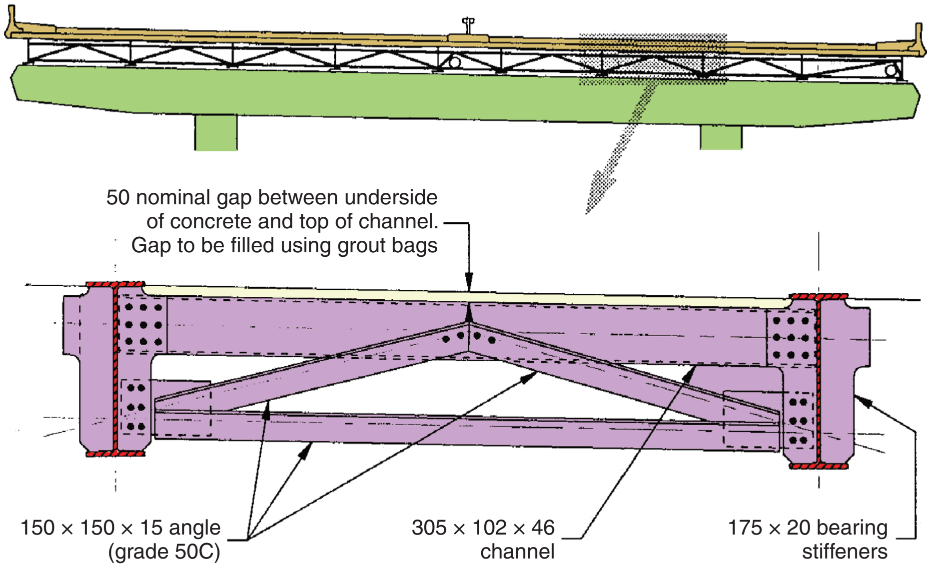

Figure 2.9 – Permanent bracing details

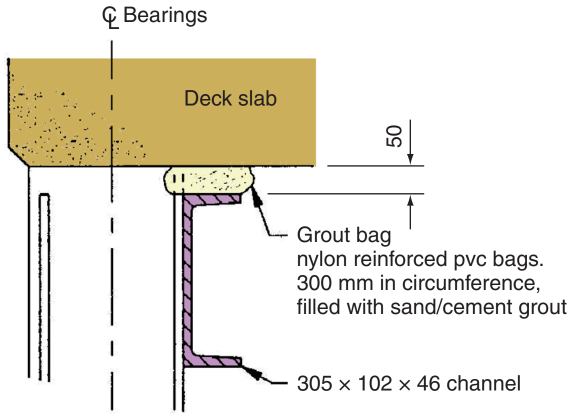

Figure 2.10 – Grout bag detail

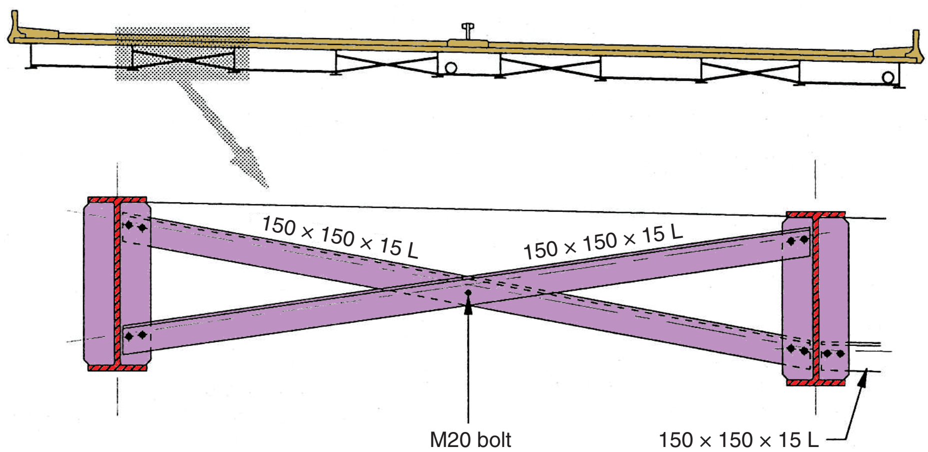

Figure 2.11 – Temporary bracing details

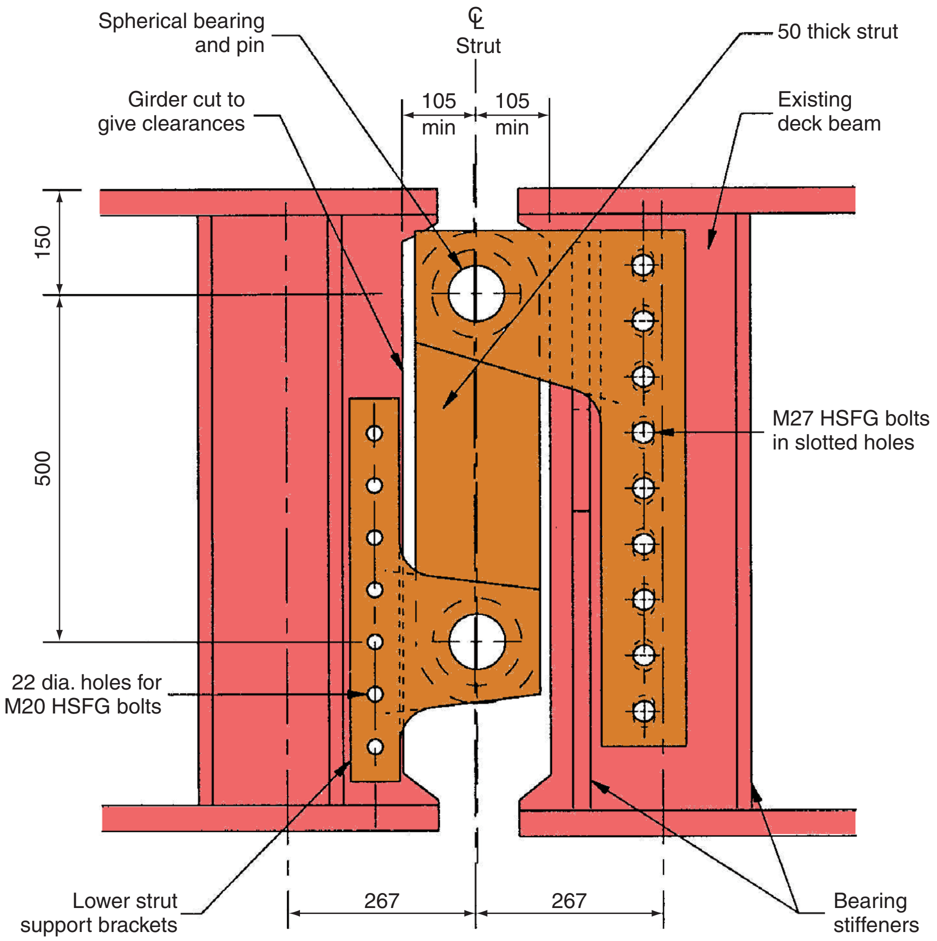

Figure 2.12 – Pinned connection unit

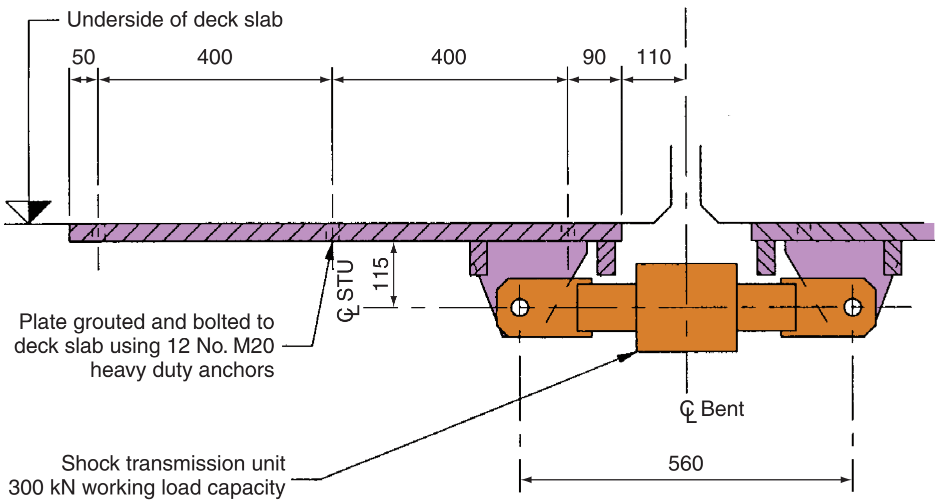

Figure 2.13 – Shock transmission unit

Temporary guides to bearings

The removal of the panel walls released the transverse restraint to the deck. Until the deck had been lowered on to the transversely fixed bearing on the temporary supports, there would have been no transverse restraint to the deck. This problem was overcome by fixing temporary guides to selected existing bearings prior to demolishing the last panel walls. The guides had to operate both before and during the jacking operation until the bolts of the fixed temporary bearings were tightened.

Angles with machined faces were therefore bolted to the deck beams, and brackets, which were clamped to the existing bearings, fitted against them to provide guiding faces which would slide longitudinally and vertically while transmitting the required transverse loads. The sliding surfaces were greased, and the brackets were released in a controlled manner at the end of the jacking operation to transfer the transverse forces from the existing bearings to the temporary fixed bearing.

Dynamic behaviour

During the design there was concern that there may be dynamic problems with the behaviour of the deck when temporarily supported. With traffic crossing the structure it was possible that a ‘springboard’ action of the deck cantilevers may cause excessive vibrations.

Dynamic analyses were carried out using a special computer program which allowed the effects of a vehicle crossing the deck to be modelled. Results were given in terms of deflections, forces and reactions plotted against time. A typical plot of temporary support reaction and cantilever deflection is shown in Figure 2.14 which showed that the deck could uplift at the temporary jacking point and considerable vibrations would occur at the cantilever tip.

In order to avoid the problem, a pin was introduced which linked the two deck ends together. The change to the dynamic behaviour is shown on the other plots in Figure 2.14. The pinned connection units (PCUs) were fixed to the deck beam ends by substantial brackets which were designed to avoid the new bearing stiffeners. Slotted holes were used in one of the brackets to cater for construction tolerances and deck beam deflections between installation prior to jacking and bolt tightening after jacking. The brackets had to be appropriately shimmed to allow for lack of alignment between the pairs of deck beam ends. Since the deck beam webs were not truly vertical, and to cater for small torsional rotations of the deck beams, spherical joints were chosen between the vertical link and the brackets (see Figure 2.12). These required periodical greasing while in use, and tubes were led away from the joints to more accessible greasing points. The PCUs were subjected to a full scale laboratory loading test prior to installation.

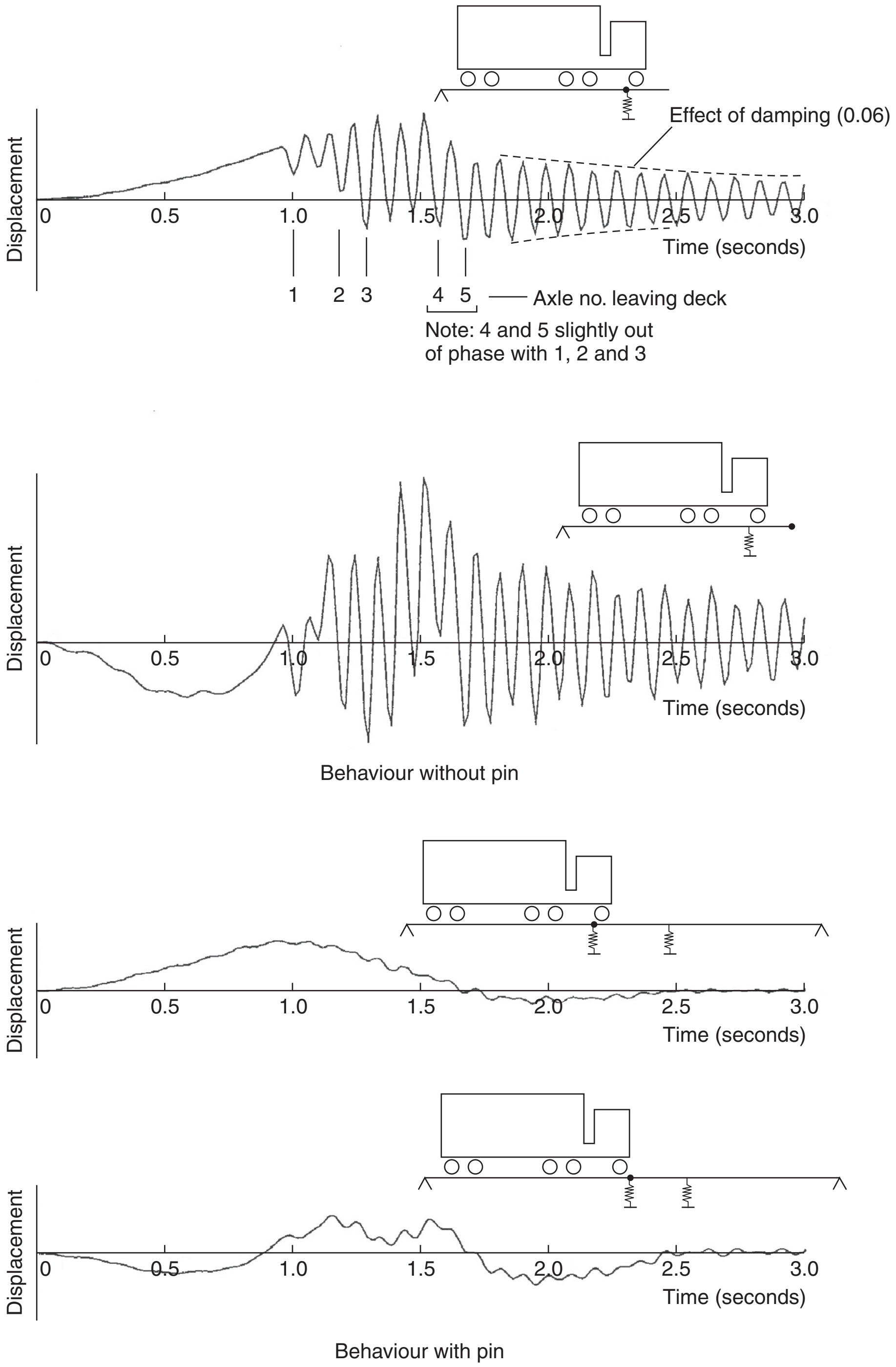

Figure 2.14 – Dynamic analysis results

The other possible solution of changing the support points to reduce the cantilever lengths was rejected because of the reduced access space for work on the crossbeam and deck. This is discussed below.

Viaduct articulation

Although the viaducts comprise simply supported spans, their articulation is unusual and also highly indeterminate. The steel deck beams rest on steel on steel sliding bearings which were initially greased during construction but are known to now have high friction coefficients approaching unity. The decks are stabilised by the reinforced concrete panel walls which connect the deck slab directly to the crossbeams. Transverse forces are directly transmitted from the deck to the crossbeam while in the longitudinal direction the panel walls flex to accommodate temperature movements but are capable of transmitting any longitudinal forces which are in excess of the bearing friction forces.

A single span in a long length of similar spans acts in a similar manner to continuous welded rail, as the adjacent spans offer restraint and temperature movements are taken up in the span module by sliding, even on the high friction bearings. When the articulation is altered by introducing free sliding bearings then the modular system no longer applies and interaction occurs. If the continuous welded rail analogy is considered then the effect is similar to cutting the rail as large movements occur where previously there were none.

While some movement was expected, the amount was considered unpredictable although attempts to model the effect using non linear analyses were later carried out with some success. When the structure was monitored on site the expansion and contraction lengths were found to be equivalent to the thermal movement of four to five span lengths which under extreme temperature conditions could have overstressed the temporary supports. In order to control the movements to within acceptable limits, restraints were added to the deck beam ends to give the continuity which had been removed by the articulation modifications. These consisted of tie bars to transmit tensile forces during contraction and packs between the bottom flanges to transmit compression forces during expansion.

Erection and installation of supports

The deck jacking was to be carried out with the motorway closed to traffic over a weekend. The restricted period available for closure of the motorway required the jacking procedure to be carried out as efficiently as possible. In order not to overstress the deck slab or the K-Bracing, the deflections of the deck beams had to be kept to within ± 1 mm of the adjacent beams. Thus at each stage of jacking the deflections of each of the 20 deck beams had to be read. It would have taken too long to read each of 20 dial gauges positioned at each deck beam, therefore, the jacking procedure specified the use of linear variable displacement transducers (LVDTs) at each beam, led back to a central console adjacent to where the 40 jacks could be pressurised. In the event, this procedure worked very well.

In jacking up the deck, a nominal gap had to be achieved over the existing bearings to prevent live load closing the gap and to permit the existing bearings to be removed. As the deck was changing from being simply supported on the existing bearings to cantilevering beyond the jacking points, the required lift at the jacks was greater than the required gap over the existing bearings. This was calculated, but as it involved the concrete deck, acting compositely with the deck beams, going into tension, the accuracy of the calculation could not be guaranteed. By jacking up the deck at a rehearsal prior to the final operation, the lift at the jacks, and hence the thickness of the spacer plates required over the temporary bearings, could be determined. This then saved time during the final jacking, by streamlining the operation of inserting the spacer plates. The other equally important reasons for the rehearsal was to practise and time each operation to ensure it could be done in the time available, and to ensure that the deck could be lifted satisfactorily on the jacks without tilting from side to side, or losing synchronisation between the lifting of the two deck ends. In the event, the jacking rehearsal was to prove worthwhile in ensuring that for the final operation the motorway was reopened in time, prior to the heavy Monday morning traffic.

The levels of the temporary steel supports were carefully monitored as the deck load was applied to ensure that the structural behaviour was as anticipated.

The procedure for the jacking had to be laid down carefully for the contractor to follow, to ensure that each item described above was completed at the appropriate point during the jacking operations. There were many meetings with the contractor and specialist jacking subcontractor to ensure that the operation could be carried out within the possession periods. This led to some changes which streamlined the operations.

Sand lorry trials

In order to ensure that the PCUs operated satisfactorily in eliminating unacceptable vibrations in the deck beams, immediately after jacking a sand lorry was run a number of times across the deck joint at differing speeds. A timber plank was positioned on the carriageway for several of the runs to stimulate an impact loading. The amplitude and frequency of the deck deflections were recorded by accelerometers, at mid span, cantilever end, and over the temporary supports.

Vibration monitoring was continued under live traffic to provide a further guarantee. The results of the monitoring were compared with the theoretical dynamic analysis of the deck before proceeding with the demolition of the crossbeam. In the event the theoretical results matched well with those in practice and the effectiveness of the PCUs was confirmed.

Demolition and reconstruction

The demolition involved the removal of $60\,m^3$ of concrete and 30 tonnes of reinforcement. There were considerable constraints of access between the temporary supports and there were safety considerations as large pieces of reinforced concrete were to be removed and the contractor wished to use machine mounted breakers.

It was decided not to remove the crosshead in pieces as this would present too many problems with handling and impose risks of damage to the temporary supports. The use of machine mounted breakers was tried by the contractor but this was found to cause large vibrations in the columns and foundations which were to be retained.

The contractor therefore used hand held breakers in conjunction with mechanical disruption devices which initially cracked the concrete.

Where reinforcement had to be retained, eg, the column starters, then water jetting was used. This was successful, though time consuming. The core concrete was found to be particularly difficult to remove because it was strong and heavily reinforced.

The contractor laid his crossbeam soffit formwork in advance and used it to collect the demolition debris. On completion of the demolition this was adjusted to line and level and the reinforcement cage assembled in-situ (see Figure 2.15). Because of the site constraints and the protrusion of the column starters prefabrication of reinforcement was not possible.

Figure 2.15 – Steel fixing for replacement crossbeam

With the side forms in place access was again restricted and the high slump 10 mm aggregate concrete was placed by pump. A trial of the concreting operation was carried out to confirm the mix suitability and to ensure that compaction could be carried out successfully. This proved to be a worthwhile exercise.

The permanent pot bearings were set at levels which allowed for elastic and creep deformation of the replacement crossbeam. The outermost bearings were set high to minimise the creep deflection loading on the K-bracing.

Dejacking

The dejacking was again carried out under a complete motorway possession. This was generally a reverse of the jacking procedure. There were a considerable number of activities to be carried out and the programme was carefully planned and executed. After dejacking, the lower connections of the K-bracing were released and rebolted to relieve load arising from any construction tolerances in the levels of the permanent bearings.

Further applications

Crossbeam replacement was always regarded as a last resort but the exercise was useful in identifying problems and allowing the costs to be realistically assessed. Compared to the other options for crossbeam repair there are clear disadvantages. However, providing a new uncontaminated structure with minimal future maintenance also has its attractions.

In reviewing the past work there are improvements and alterations which could be made to reduce the large costs and these are explained below.

Simplifications

The supports could be simplified by reducing the onerous safety requirements. Instead of a double leaf system then a single leaf could be used, as has been used previously on a similar application. The structure, however, is vulnerable if one of the legs is damaged in which case a collapse would occur. Given knowledge of site operations and the protection measures which can be introduced this may be viable but whether the increased risk is acceptable is debatable.

By incorporating bearings on jacks or only using jacks the lifting and dropping down operations could be avoided.

Foundations

Providing foundations for the temporary support was difficult, time-consuming and expensive. A scheme has been developed which uses the existing foundations for support but this would have proved very complex to construct and erect underneath the crossbeam.

Other modifications

In order to overcome the dynamic problems with the cantilevers then the support points could be moved by providing beams spanning between the temporary supports and supporting the deck from cross girders which form part of the permanent bracing. This has advantages in eliminating the use of pins connecting the two deck ends but makes access to the crossbeam even more difficult. It has been successfully used elsewhere however.

Conclusions

The removal of a complete motorway support under live traffic conditions was successfully carried out although it proved to be both a complex and expensive operation. The major considerations are the safety of the structure because of the massive economic costs of disrupting traffic on a major highway.

Some of the problems associated with the design and construction have been described and improvements have been considered. A second crossbeam was replaced some years later, however, and after due consideration, a similar method was successfully followed. The condition of that crossbeam is described in the Case Study 10.

Case 3 Panel walls

Introduction

Panel walls can be used to support the free ends of simply supported steel/concrete composite bridge deck slabs. They are of reinforced concrete and cantilever upwards from the supporting reinforced concrete crossbeam. The panel walls provide transverse restraint to the deck and, contrary to current standards, have been used to avoid the need for bearing stiffeners on the steel deck beams. Panel walls thus carry transverse skidding loads and centrifugal forces. In the longitudinal direction they flex to accommodate shrinkage and temperature movements and live load compression in the deck. Also in the longitudinal direction they resist braking, traction and skidding forces. Wind forces can also be accommodated by panel walls.

The problems

The case study examined arose while the crossbeams were being repaired for chloride attack resulting from road salts leaking through the bridge deck joints, impregnating the panel walls and crossbeam, and causing rebar corrosion and delamination.

Problems encountered on the panel walls included diagonal cracking resulting from transverse shearing forces derived from shrinkage of the deck slab concrete. Further problems arose whereby during construction the large longitudinal bars in the crossbeam had been misplaced, preventing correct positioning of the vertical bars in the panel walls. The latter bars had then been cranked immediately above the crossbeam to bring them into position and provide the correct cover to the panel walls (see Figure 3.1). The result was that at the bottom of the panel wall the required lever arm was not provided by the vertical bars so that the required bending resistance of the panel wall was far from being achieved.

Figure 3.1 – Panel wall vertical reinforcement cranked into position originally

Damage

Many of the vertical bars and some of the horizontal bars in the panel walls were severely corroded with pitting corrosion towards the bottom of the panel walls with greatly varying loss of section, 40% loss being quite common. Consequently much of the area of the panel walls was delaminated. Sometimes the corrosion of the vertical bars encroached within the cover zone on top of the crossbeam (see Figure 3.2).

Figure 3.2 – Concrete spalling due to corrosion of panel wall reinforcement

Concrete removal

Since the panel walls were only 150 mm thick, the concrete was water-jetted out through the full thickness of the wall over areas which were delaminated on one or both sides (see Figure 3.3). The deck and crossbeam and adjacent panel wall were screened to prevent water-jetting damage and strict procedures were followed to ensure the safety of the live traffic above. Calculations had shown that by working on a half length of panel wall at a time the deck was adequately supported and no traffic restrictions were required. Further calculations showed that across the width of the dual three lane motorway and hard shoulder, within which there were eight panel walls, adequate provision would remain to carry braking forces if a limited number of half panel wall widths were removed at one time. Repair sequence drawings were produced showing the order in which panel walls were to be repaired (see Figure 3.4).

Rebar repair

Chlorides remained in the concrete in non-delaminated areas but ongoing corrosion in these areas would be arrested by the application of cathodic protection over all the panel walls. Assessment calculations were carried out to determine the average loss of section which could be tolerated on the vertical bars and sufficient bars were repaired to satisfy that requirement. Repairs were achieved by grinding off the pitted areas to a flat surface and refilling with weld metal, strictly following prior established weld procedures based on chemical analysis of the bars. Tensile tests had been previously carried out in the laboratory which proved the adequacy of such a repair technique.

Figure 3.3 – Concrete removed from panel wall by waterjetting

Where the depth of repair exceeded 40% of the bar diameter, grinding would have been unsuitable and the replacement weld depth too deep. In these situations additional lengths of bar were fillet welded alongside the damaged bar to replace the strength, all corrosion products having been removed from the damaged bar by the water jetting (see Figure 3.5). (Where the pitting occurred close to, or within, the cover zone of the crossbeam, the damaged bar was cut out and a replacement length butt welded in). Calculations were carried out to determine the length of fillet weld required to transfer the force in the bar. Where possible, double fillets were provided. High bending stresses due to the eccentric reaction are generated in a single fillet weld between bars and a significantly longer weld is required.

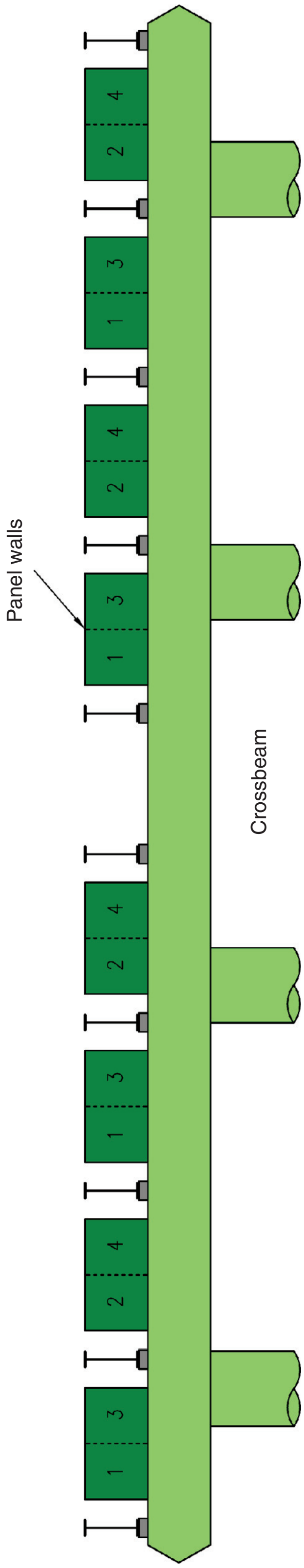

Figure 3.4 – Typical panel wall repair sequence

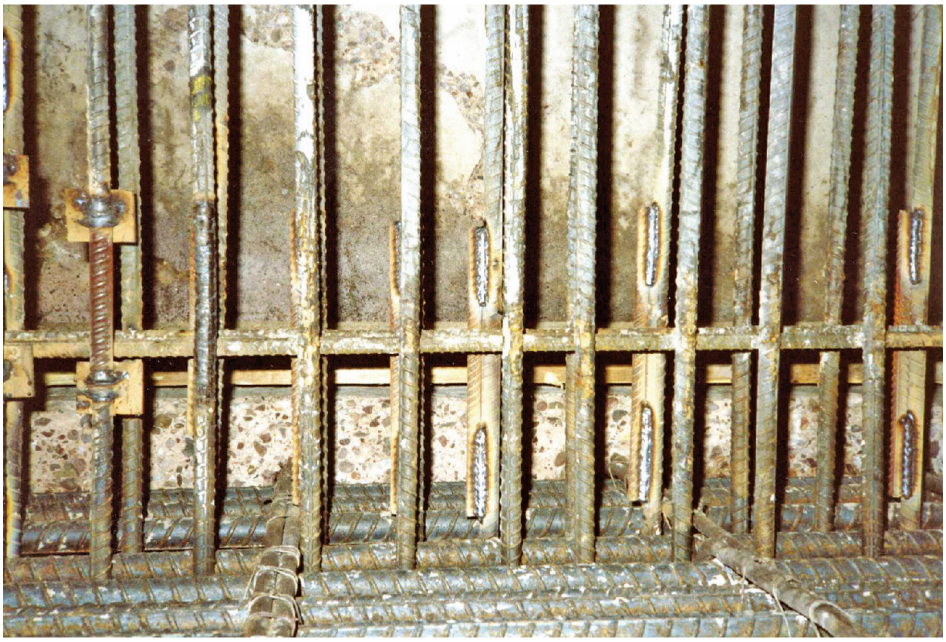

Figure 3.5 – Fillet welding, weld filling, and butt welding of panel wall reinforcement

Crossbeam break-out

Where the weld repairs encroached into the cover of the crossbeam, but the crossbeam itself did not require extensive repairs and consequently was not propped, water-jetting was not allowed owing to the risk of accidentally removing too much structural concrete from the crossbeam. Break out was then by hand-operated impact hammers. Such a technique is now subject to special controls, due to concerns over white finger.

Strengthening

Strengthening of the panel walls at the cranked vertical bars consisted of inserting additional L-shaped bars at an increased lever arm with the bob returned under the main bars in the top of the crossbeam. The panel wall was then reconcreted at greater thickness to each side over the lower section, to provide adequate lever arm to the cranked bars. The L-shaped bars did not have sufficient anchorage strength to act as full vertical reinforcement, but were nevertheless able to control crack widths in the extended cover zone to the cranked bars. The application of cathodic protection further reduced the risk of corrosion. Only the lower section of the panel wall was thickened to retain flexibility under deck temperature movements.

Hinge repair

Breaking out the top of the panel wall immediately under the deck soffit was avoided where possible as this was difficult to reconcrete. It consisted of a hinge formed by chamfers to each top corner, with a dowel bar connection. Normally this was left in place and the concrete below broken out, leaving an inclined soffit which was easy to concrete against. Where the hinge required replacement, repair concrete was squeezed into place between the chamfer formers.

Case 4 Crossbeam repairs

Introduction

Crossbeams suffered chloride attack resulting from road salts leaking through the bridge deck joints, impregnating the concrete, and causing rebar corrosion and delamination.

The crossbeams varied from 1 to 2 m in depth, were 1.67 m in width and were up to 35 m long, normally supported by between 2 and 4 circular reinforced concrete columns.

The main reinforcement comprised high-yield deformed steel bars of 25–35 mm in diameter at, as close as, 75 mm centres in the top and soffit faces, with plain round mild steel rectangular links or lapped hairpin links between 12 and 20 mm in diameter, together with 20 or 25 mm plain round mild steel torsion bars surrounding the reinforcement cage, and lapped on the side faces.

Rebar repair

Generally the corrosion of the main bars was not severe. Apart from the two crossbeams which were replaced, only on one crossbeam was there a need to supplement the main reinforcement. Elsewhere there was either sufficient capacity for the average loss of section to be accommodated, or the loss of section could be addressed by weld filling as described in Case Study 3.

The torsion bars were severely corroded particularly at the top corners of the crossbeams, some bars being corroded almost entirely through the cross section. Corroded lengths were cut out and new lengths butt-welded in (see Figure 4.1).

The links were also severely corroded, particularly at the bends (see Figure 4.2), and replacement lengths were welded in. Corroded ends of the hairpin links were strengthened by fillet welding additional horseshoe bends to the existing U-bend, since access was not available to cut these out and butt weld behind the closely spaced main reinforcement.

Figure 4.1 – New length of torsion bar butt welded into position

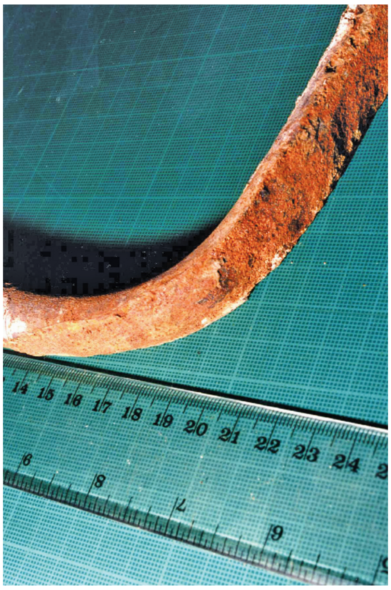

Figure 4.2 – Corrosion of link bar at bend

As cathodic protection was to be applied to the crossbeams, only the delaminated areas were broken out for repair by proprietary flowing concrete to behind the main reinforcement. This possibly left some pitting corrosion in the torsion bars in non-delaminated areas which were not repaired, but there was sufficient strength in the concrete section alone to carry the required torsion.

Unpropped repairs

Crossbeams with only small area of delamination were repaired first so that the small repair areas could be repaired unpropped before they further increased in size to the extent that propping would be required during repair. The larger of these areas were divided into two to four portions depending on size and location, and the portions repaired in sequence (see Figure 4.4). Breakout was by hand-operated impact hammers as water-jetting would have significantly increased the risk of accidental overbreak. (Concerns over white finger now require strict controls). Within small areas of delamination there was a much reduced chance of reinforcement repairs being required.

Figure 4.3 – Temporary truss support

Propping

Crossbeams with larger areas of delamination were propped for repair by prefabricated trusses (see Figure 4.3), or by plate girders, (see Figure 4.5) supported on steel trestles around the concrete columns (see Figure 4.6). These were designed to carry the full dead and live load to cater for potential crossbeam failure during repair under live traffic. The dead load was jacked into the supporting steelwork, and the loads in the jacks periodically checked (see Figures 4.7 and 4.8). The stiffness of the steelwork being significantly less than that of the crossbeam, only some 20–30% of the live load was carried by the steel work. However, this proportion of the live load, together with the relief of dead load, meant that half of the crossbeam width was able to carry the residual forces in the crossbeam. This enabled half the crossbeam width to be repaired at a time.

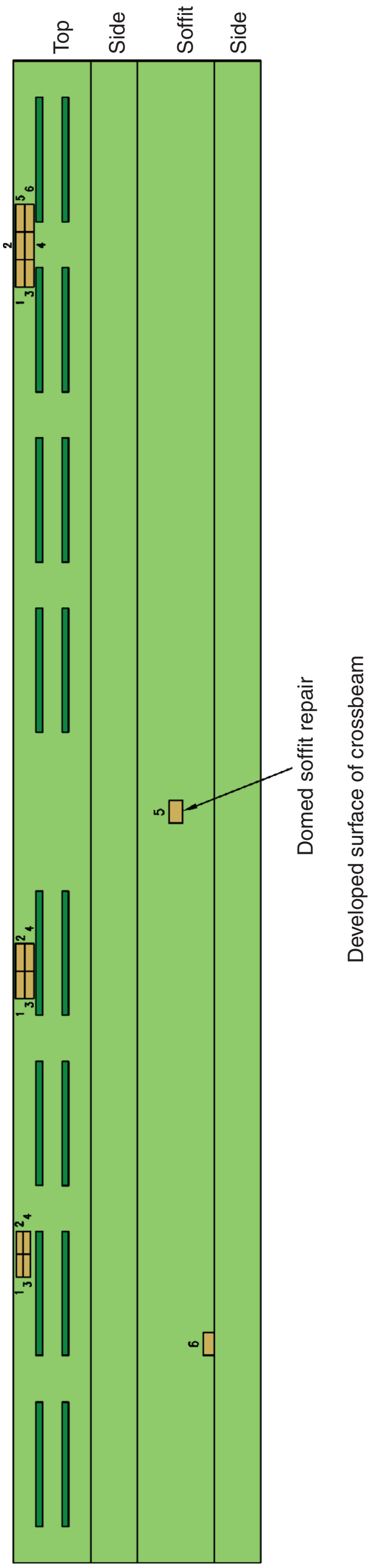

Figure 4.4 – Typical unpropped crossbeam repair sequence

Figure 4.5 – Temporary plate girder being erected

Figure 4.6 – Temporary steel trestle

Figure 4.7 – Temporary jacking beam

Figure 4.8 – Jacking beam jacks

Faying surfaces

The trusses were partially prefabricated and then linked together during erection. The steelwork was galvanised and the joints formed with HSFG bolts. It was important that the design value used for the friction of the faying surfaces was achieved on site by the needle gunning process. Test samples of bolted joints were put in a laboratory tensile machine to prove the requisite friction had been achieved, and proven samples were put on display with which to visually compare the roughness of the actual surfaces being prepared on site.

Soffit repair

The delaminated areas of the crossbeam were then repaired in a calculated sequence. (see Figure 4.9). The soffit was first repaired in discrete lengths between the jacking positions. The concrete was broken out to behind all layers of soffit reinforcement (see Figure 4.10) so that an inclined soffit was exposed which could be reconcreted by pumping flowing repair concrete through the soffit formwork close to the centreline of the beam with minimised risk of creating air voids. Insufficient gradient could be provided by a soffit sloping between the reinforcement layers. Breaking out was by water-jetting horizontally across the beam width, particular care being taken not to go beyond the beam centreline. The concrete, once it filled the soffit, was brought slightly up the side of the beam to create a head within the soffit area.

Figure 4.9 – Soffit break out

After the concrete had gained strength, sample holes were drilled in the soffit and a boroscope used to check for the absence of air gaps. The jacks were later moved to alternative locations to enable the remainder of the soffit to be repaired. The soffit was preferably repaired first so that the construction joint on the side of the beam could generally be formed by concreting from above. Discrete delaminated areas towards the centre of the crossbeam soffit were broken out from the side face, both to enable horizontal water-jetting, and to avoid the formation of air gaps. In water-jetting normal to the face of the concrete, it would have been very difficult to avoid creating deep holes in the broken out surface of the concrete.

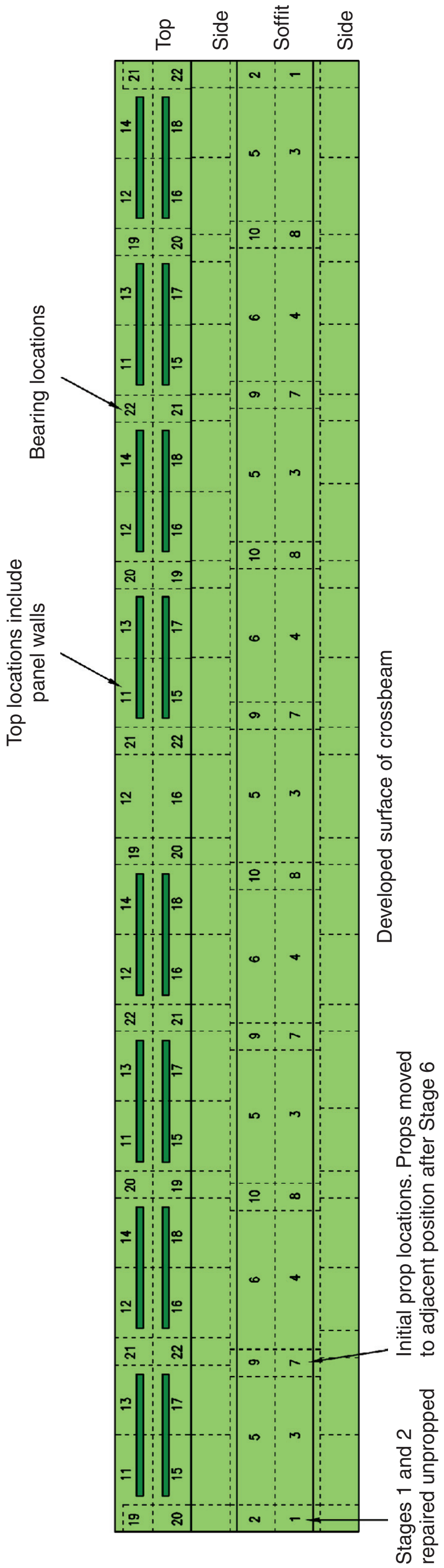

Figure 4.10 – Typical propped crossbeam repair sequence

Domed soffit repair

Some crossbeams over highways could not readily be propped and a method was developed for repair without propping of small areas of delaminated soffit which did not extend to the face of the beam. The concrete was broken out by hand to behind the outer layer of reinforcement to form a domed soffit to the broken out concrete. A vertical plastic tube was inserted through the formwork just short of the top of the dome. When the concrete was pumped into the void, it overflowed through the tube, ensuring any residual air gap was limited to a tiny area at the top of the dome (see Figure 4.11). The ends of the crossbeam were repaired unpropped by breaking out to the edge.

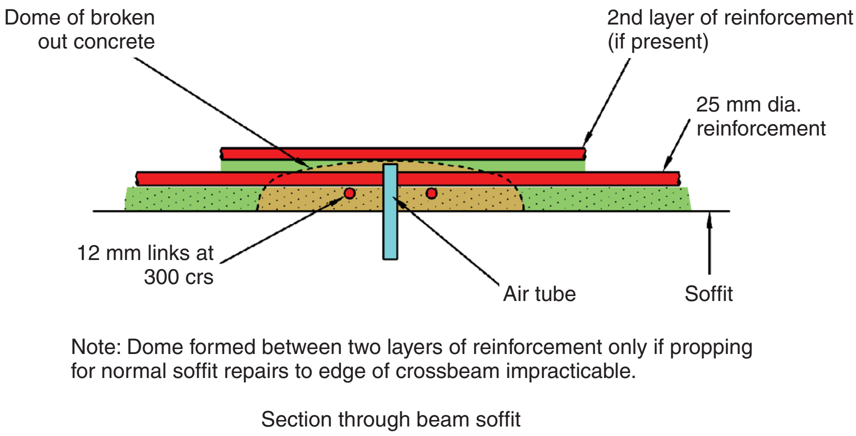

Figure 4.11 – Domed soffit repair

Top and side repair

The top and sides of the crossbeam were repaired together with water-jetting parallel to the top of the beam. The sides were easily jetted due to the absence of longitudinal reinforcement. The top of the beam only needed to be broken out behind the top layer of reinforcement, since reconcreting from above was straightforward (see Figures 4.12 and 4.13). Reduced flowing concrete was used in the top of the beam to enable the concrete to be cast to the fall of the top of the beam. Repairing under bearings is described in a separate case study.

The repair areas were separated by the bearings, by the panel walls being repaired in half lengths (see Case Study 3) and by the need to work on half the beam width. Given sufficient length between repair areas at different locations along the beam could be exposed on opposite sides of the beam at once. This was generally avoided where possible to avoid errors – it was easier to supervise the procedure if only one side of the crossbeam was being worked upon at any one time.

In some crossbeams the repair areas were sufficiently small in the bays between some columns to be suitable for repair without propping, whereas in other bays the repairs were sufficiently extensive to require propping. For such cases of partial propping, rules were developed to safeguard the unpropped bays. The hogging moment zone on the top of a beam beyond the column adjacent to an unpropped bay was broken out by hand impactor and not by water-jetting, in case an excessive amount of concrete was accidentally broken out. Repair areas in this zone were restricted in size in the same way as in unpropped areas.

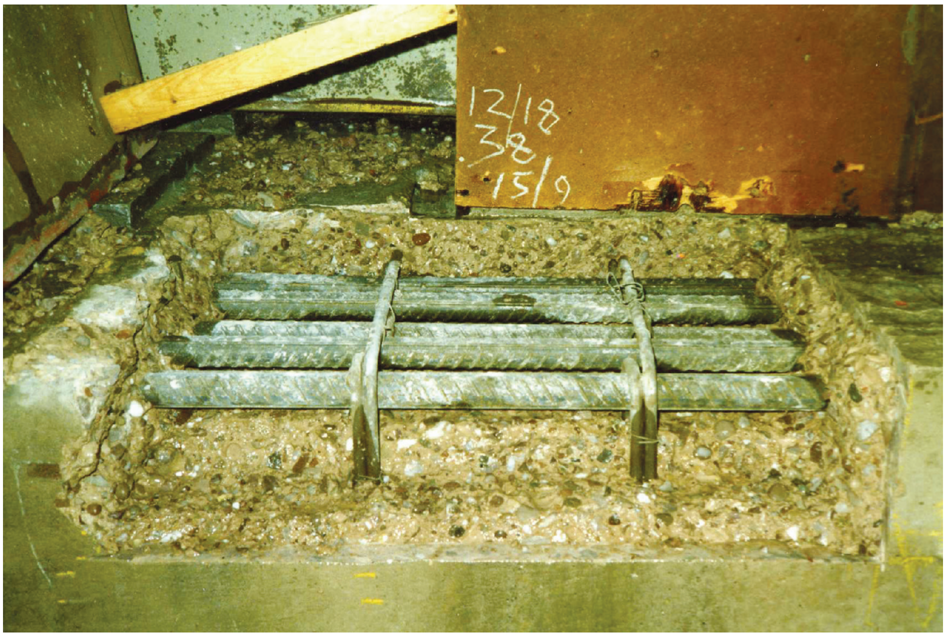

Figure 4.12 – Break out to top of crossbeam

Figure 4.13 – Flowing concrete trial for crossbeam top

Case 5 Cantilever repairs

Introduction

Crossbeam cantilevers suffered chloride attack resulting from road salts leaking through the bridge deck joints, impregnating the concrete, and causing rebar corrosion and delamination. There were in excess of 1,000 crossbeams and therefore over 2,000 cantilevers, most of which were delaminated. Given the lack of redundancy in the cantilevers, there was concern as to their shear strength in delaminated condition and the number of elements involved meant that it was cost effective to carry out model testing to determine this delaminated strength.

Model testing

Some 24, one-third scale models were built representing the most common configurations of the cantilevers. Some were in pristine condition, some with the concrete cast or scabbled flush with the main bars and some were repaired flush with, or behind, the reinforcement (see Figure 5.1). Corrosion was represented by notching the main bars at 100 mm centres through 10, 20 or 30% of the cross-sectional area, and the corners of the stirrups through 30, 60 or 90% of the cross-section. The beams were loaded to failure and the results analysed (see Figure 5.2). Tensile tests of the notched bars indicated that strain hardening occurred at the notch, so that the loss of strength was by no means proportional to the loss of section. However, the loss of bond strength due to delamination was highly significant.

Figure 5.1 – Model beam repair

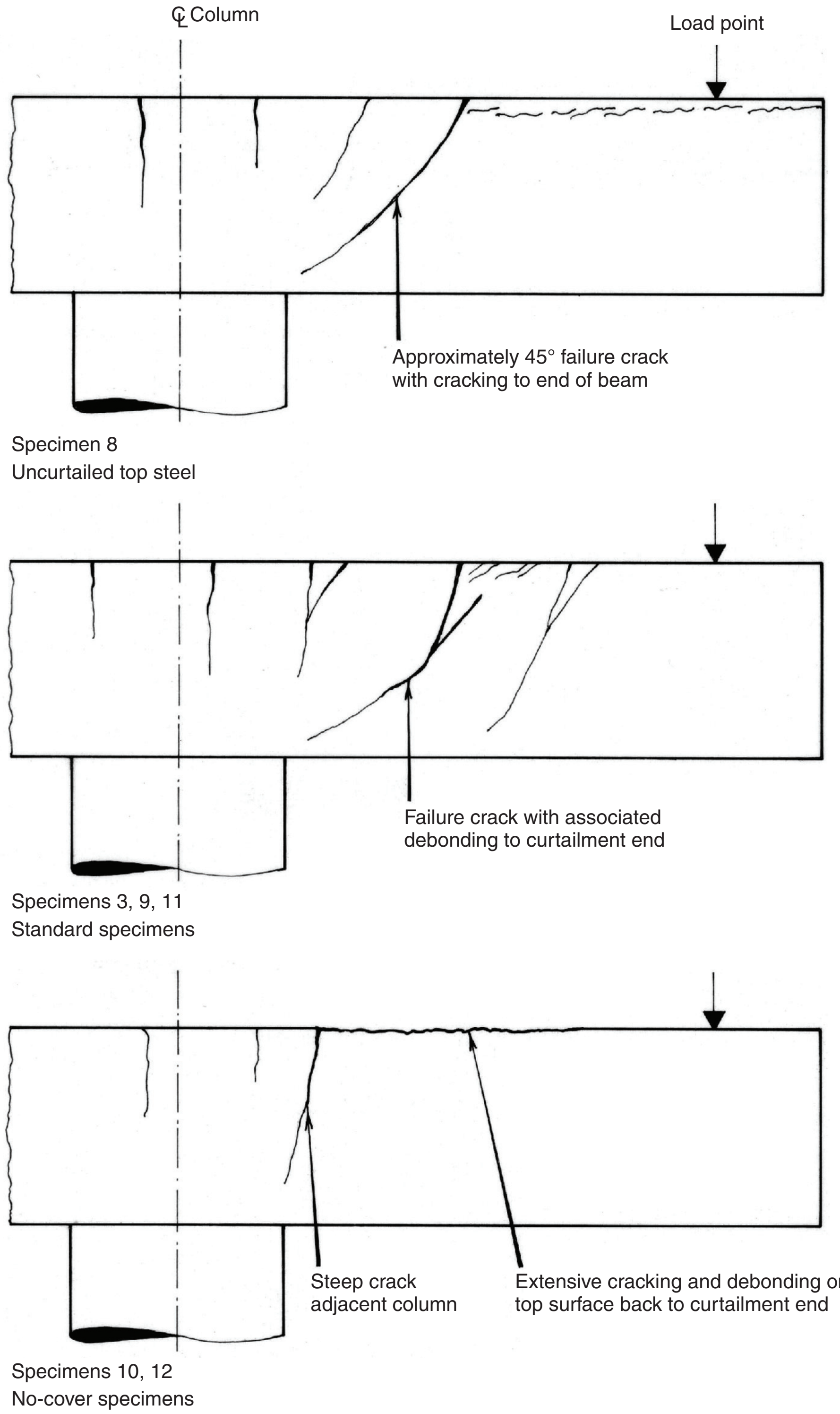

Figure 5.2 – Failure modes – cantilevers

Pull-out bond tests at half-scale were then carried out to determine the bond strength for different conditions, such as full or flush cover, bar spacing, number and spacing of stirrup legs, scabbling back the cover, or repair (see Figure 5.3). The bond values so obtained were then used in back analysis of the beam tests using Regan's method.

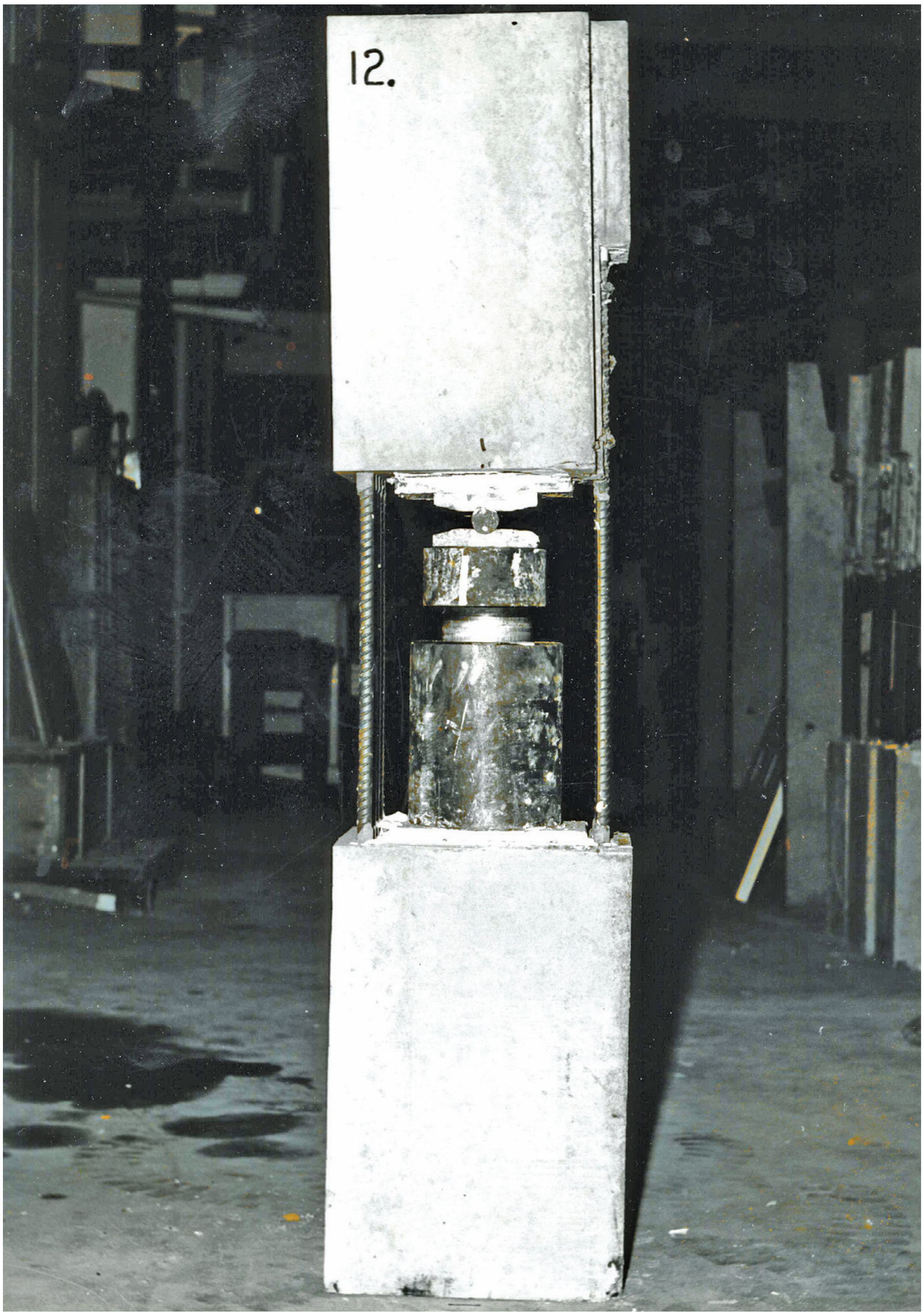

Figure 5.3 – Bond test

Regan's method

This method was developed by Professor Paul Regan, who carried out the testing, to calculate the force in main bars due to the combination of bending and shear. This force had then to be resisted by the delaminated bond strength of the main bars in the top of the cantilever between the failure shear crack and the outer end of the bars. Having thus confirmed the validity of this method, and of the values adopted for the delaminated bond strength, the deteriorated strength of most of the cantilevers on the viaducts could then be assessed.

Load restriction

This approach showed that those cantilevers of a certain type which were heavily delaminated would be significantly overstressed under abnormal loads over 100 tonnes on the hard shoulder. Henceforth permits for loads over 100 tonnes to travel along the viaducts were conditional upon not using the hard shoulder. This safeguarded the integrity of the cantilevers until those of that type with extensive delamination could be repaired.

Repair

The cantilevers were repaired by the methods described in Case Study 4. Normally since either the repairs were carried out unpropped, in which case the areas delaminated were small, or the repairs were carried out with the crossbeams propped, in which case the risk of beam failure was low, the weakness due to delamination of the areas alongside those being repaired was ignored. However, in view of the lack of redundancy therein, the analysis of the most vulnerable cantilever type later took into account the reduced strength of the delaminated areas using Regan's method and the bond values derived from the laboratory testing. That type of cantilever had only a single layer of main reinforcement with half the bars curtailed part of the way out along the cantilever, thereby reducing the anchorage bond available to provide support beyond a potential shear crack. By that time the top surface of a number of cantilevers had become predominantly delaminated.

Repair diagram

Following analysis of the delaminated strength, diagrams were prepared, indicating the areas on the cantilever tops permitted to be broken out in sequence (see Figure 5.4). At the beginning of the sequence only small areas could be removed due to the extent of delamination, while towards the end of the sequence larger areas could be removed as a result of the increased strength of the areas by then repaired. The locations, as well as the size, of the repair areas were critical.

Monitoring

Some cantilevers, not the most vulnerable type, had not been repaired by the time hard shoulder running was required for deck repairs, and these were resurveyed, then monitored weekly for increased delamination and for potential shear cracks during the period of regular hard shoulder running.

Cathodic protection

As with the remainder of the crossbeams, cathodic protection was applied to arrest further rebar corrosion within the non-delaminated areas impregnated with chlorides. Concrete was broken out a little way beyond the actual delaminated area for repair in case progression of the delamination occurred before the cathodic protection could be applied.

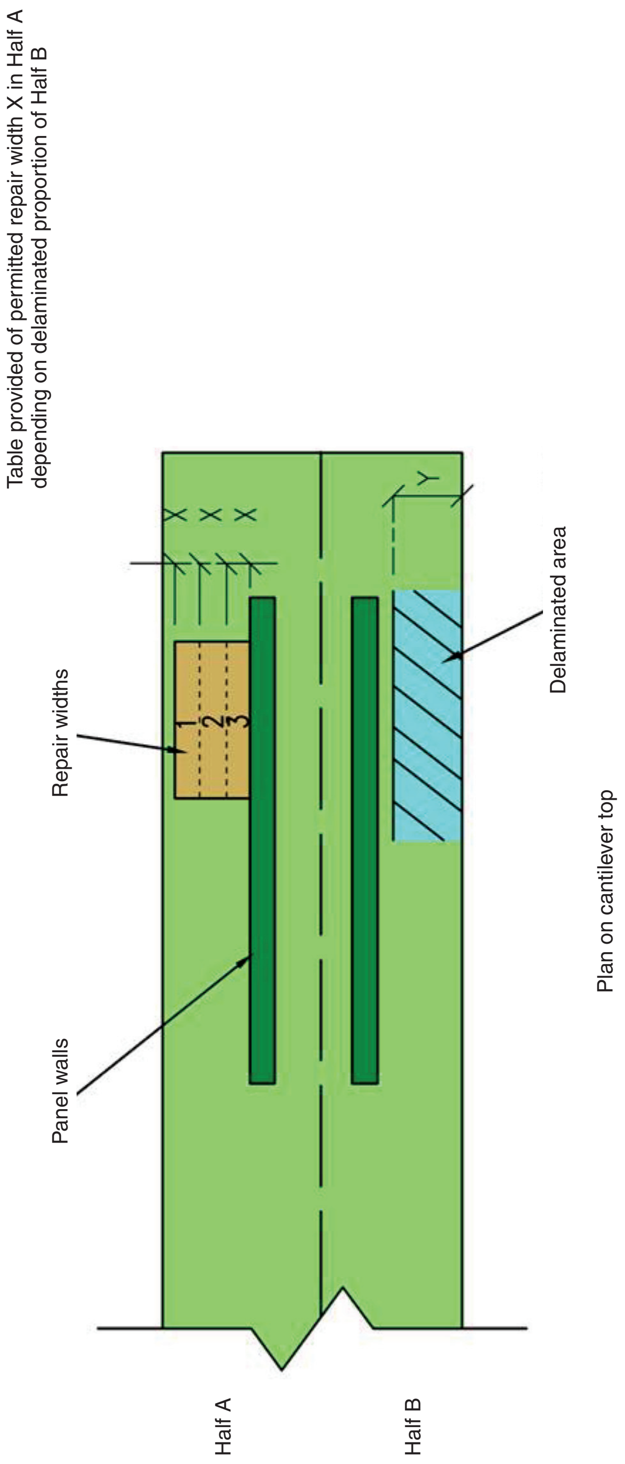

Figure 5.4 – Cantilever repair diagram

Case 6 Repairs below bearings

Introduction

This case study describes the repairs under the deck beam bearings to the crossbeams covered in Case Study 4. The bearings generally comprised a simple steel rocker with the bottom flange of the steel deck beam sliding directly on the top steel plate of the rocker. Graphite lubricant had been applied to the sliding surfaces during construction. The bearings were supported on grouted plinths.

Repair technique

The crossbeams were repaired under bearings where delamination against the plinth occurred on at least two sides of the bearing to an extent that delamination was likely to have extended beneath the steel bearing plate. The deck beam was supported under the top flange against the top of the web by a steel A-frame lifted on jacks either side of the bearing plinth (see Figures 6.1 and 6.2). The bottom of the deck beam web was strutted against the adjacent concrete panel walls for stability and the dead load in the deck beam transferred to the jacks. The bearing holding down nuts were released and the bearing clamped to the bottom flange of the deck beam. The plinth was water jetted out, as was the concrete in the top of the crossbeam, jetting from the crossbeam side between the jacks.

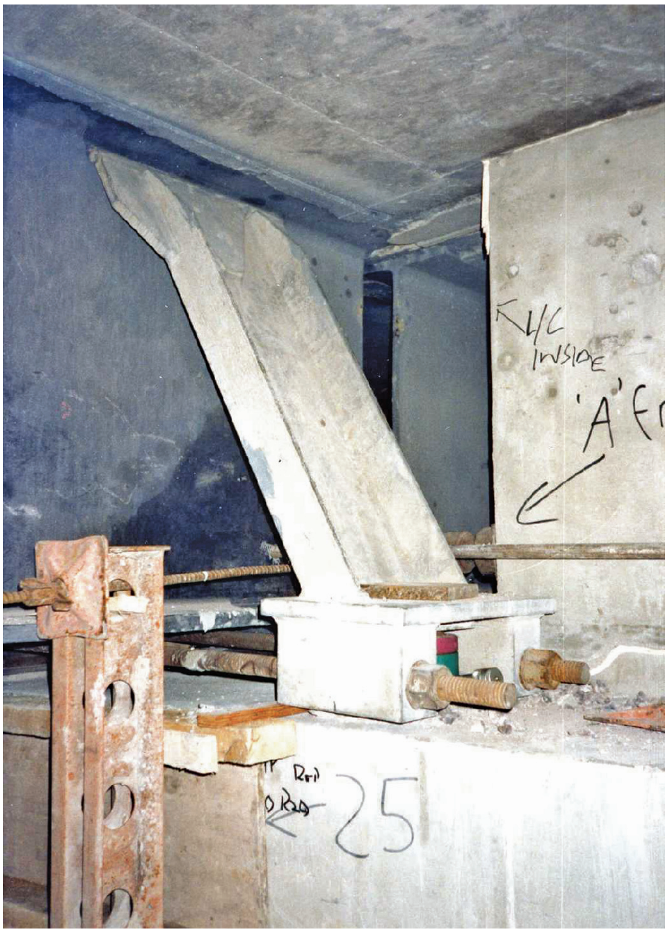

Figure 6.1 – A-frame for repair below bearing

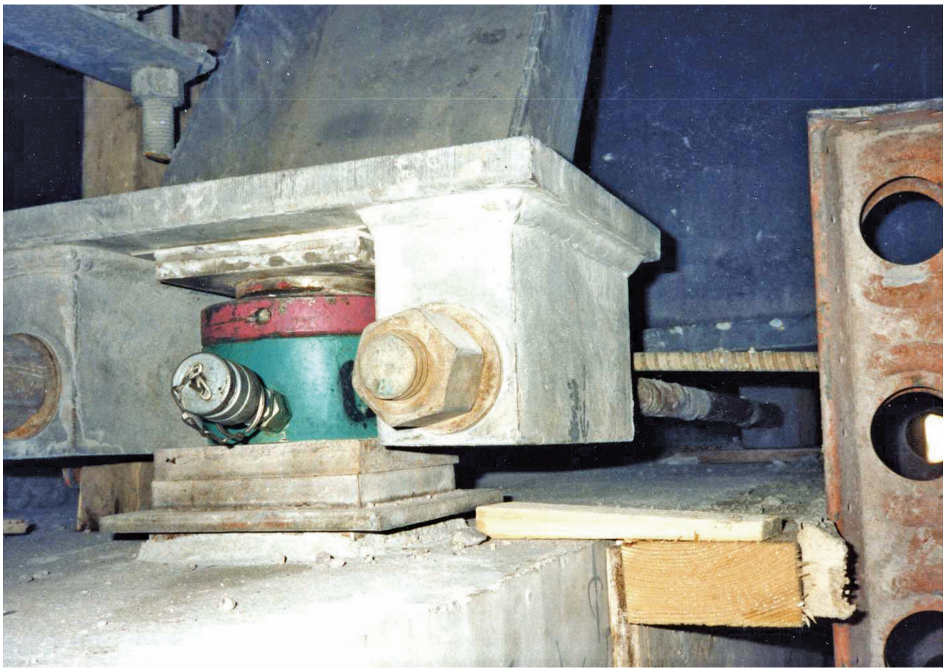

Figure 6.2 – A-frame jack

Anti-splitting mat

As this involved a significant area of breakout, repairs under bearings were only carried out with the crossbeam propped. The only transverse reinforcement in the crossbeam top continuous across the beam was the torsion bar and these were at 300 centres. This meant that between the broken out concrete under the bearing and the jack position, there might be no transverse reinforcement to prevent splitting of the concrete under the jack. As the repairs were carried out under live traffic, the precaution was taken to provide a mat of anti-splitting reinforcement under the jack positions during concrete repairs to the areas either side of the bearing, prior to the repairs under the bearing (see Figure 6.3). (The jacks were in a more vulnerable position close to the side of the crossbeam, so that the transverse Macalloy ties of the A-frame could pass beneath the deck beam in front of the bearing).

Reconcreting

As with the crossbeam repairs, the repair area under the bearing did not encroach beyond the half-width of the crossbeam. The break out to behind the top layer of reinforcement was reconcreted, then the plinth regrouted and once it had gained sufficient strength the jack pressure was gradually released to transfer the deck beam load to the bearing and plinth. Deflections were carefully monitored on jacking and dejacking. The holding down nuts were retightened.

Bolt replacement

Where the holding down bolts were corroded they were replaced. This involved breaking out the concrete of the crossbeam locally to a greater depth to permit removal of the bolt.

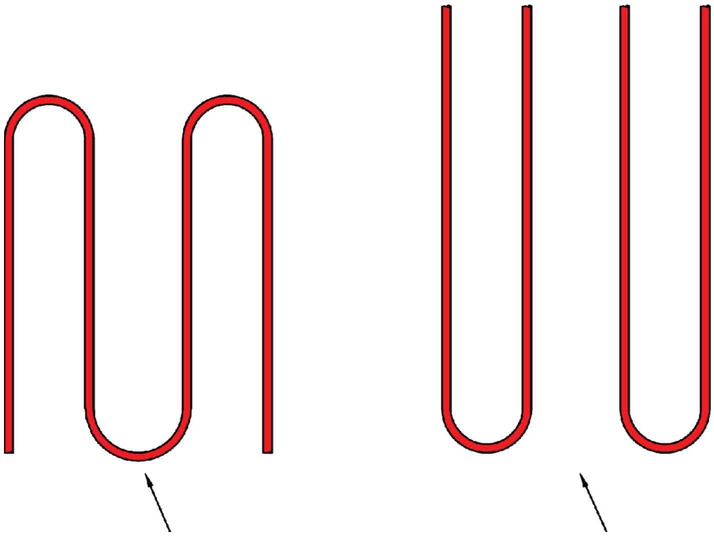

Figure 6.3 – Anti-splitting reinforcement mat

Bearing stiffeners

Generally, due to the presence of the panel walls, bearing stiffeners were not provided on the universal deck beams. However, on the longer spans the deck beams were plate girders and it had been found necessary to fit bearing stiffeners to these. For these locations A-frames were designed with slots to accommodate the transverse stiffeners.

Holes in web

In a few locations there was not sufficient depth below the bottom flange of the deck beam to take the A-frame ties, so holes had to be drilled in the deck beam webs through which to pass the ties. Strengthening washers were welded to the web before drilling the holes through them.

Skewed beams

Some deck beams were skewed, requiring a skewed A-frame design. This produced a force at the top of the A-frame longitudinal to the deck beam which was resisted by welding a steel rod to the web/top flange which butted against the end of the bearing rod at the top of the A-frame.

Top flange weld

The plate girders required the web/top flange weld to be strengthened to carry the vertical load on the A-frame.

Outer deck beams

There were slight differences for the outer deck beams. With a jack beyond the deck beam, there was insufficient anchorage in the crossbeam reinforcement to support it, and concrete was broken out along the outer edge of the crossbeam to enable hooked bars to be cast in, which would provide adequate end anchorage below the jack. The concrete below the bearing was then broken out from the end of the crossbeam, thereby avoiding exposing the newly provided hooked bars. Anti-splitting mats were not then required below the jack positions.

Narrow A-frame

For the shorter spans, a narrow A-frame was required at the outer crossbeam to enable the jack to fit on the end of the crossbeam. For the longer spans and therefore larger bearings, a narrow A-frame could not be used, and a concrete nib extension was built on to the end of the crossbeam to accommodate the jack (see Figures 6.4 and 6.5).

Strut and tie

At the outer deck beam, there was only a panel wall on one side. In addition to the provision of a strut, a Macalloy tie was therefore drilled through the deck beam web and passed round the far end of the panel wall to stabilise the deck beam web while it was supported on the A-frame.

Plinth replacement

The A-frames were also used where the bearing plinths required replacement even if crossbeam repairs were not needed below.

Figure 6.4 – Breakout for addition of concrete nib to carry wide A-frame

Figure 6.5 – Concrete nib reinforcement

Case 7 Voided crossbeams

The problem

When crossbeams were being repaired for road salt impregnation, corrosion and delamination, water-jetting the top of a propped crossbeam exposed a hole in the concrete below the top layer of reinforcement. This appeared to give way to a void. Drilling small holes in the top of the crossbeam revealed eventually a void 2 m in length, 600 mm at its widest and 450 mm at its deepest, full of salt-laden water. The crossbeam was 1.67 m in width and 1.5 m deep. A hole was drilled in from the side to disperse the water. The links through the void were severely corroded, some almost completely through.

Action

The hard shoulder was immediately closed and calculations carried out to determine whether despite being propped the crossbeam was at risk of failure, either through insufficient stiffness in the propping or by failure between the jacking locations. These calculations showed that although fortunately in the original design twice as many links had been provided than were needed it was likely that chloride impregnation of the concrete surrounding the void could have led to significant corrosion of many of the remaining links, but that once the void was reconcreted there should be minimal risk of failure provided the crossbeam remained propped.

Evidence

The only external sign of there being a problem was a fine vertical crack on one side of the beam.

The cause

After being cleaned out as far as possible, the void was therefore reconcreted and the repairs to the crossbeam were continued. It appeared that the void formed below an area of three layers of main reinforcement at close centres through which vertical bars passed, also at close centres, to form the panel walls above. Apparent lack of vibration during concreting had prevented the concrete flowing through the dense mat of top reinforcement in the region of the panel walls.

Investigation

A programme of investigation was then set in motion to drill exploratory holes adjacent to the panel walls at all locations of three layers of top reinforcement within the crossbeams on the viaducts. A number of voids were found, requiring immediate propping of the worse affected crossbeams. However no void was found approaching the size of that described above. In most cases the number of corroded links in the voids was sufficiently small not to require strengthening, even allowing for links corroded in the contaminated zone around the void.

Strengthening

Where strengthening was required a scheme was developed to enable the steel propping to be removed. This would have the advantage of not requiring periodic monitoring of the jack loads, but has not yet been brought forward to detailed design stage.

Options

The proposal was to construct flanking beams alongside the crossbeam, enabling the force actions to bypass the voided zone, since repair of the reinforcement deep in the beam in and around the reconcreted void was not practicable (see Figure 7.1). Cathodic protection deep in the contaminated zone of the crossbeam would also have been difficult, requiring the drilling of many holes for embedded anodes. Removing the contaminated concrete would have been equally difficult, for while this could have been achieved in two separate halves, the full width of the crossbeam between the top and bottom mats of reinforcement was still required to carry torsion and to distribute load from the bearing locations.

Flanking beams

The flanking beams were to have been anchored into the existing beam beyond either end of the void by breaking out the top and bottom of the beam in the usual way for repair and casting in transverse dowels to carry the longitudinal shear between the existing and flanking beams.

Shear strength

Consideration was given to casting the flanking beams in the repair concrete used on the existing beam, but repair concrete has very small aggregate to assist its flowing characteristics, and there was concern that sufficient aggregate interlock would not be provided to carry the required shear through a full depth beam.

Chloride migration

There was also the risk of migration of chlorides from the contaminated zone of the existing beam into the flanking beam over the length alongside the void. A waterproofing membrane was proposed between the two beams between the top and bottom reinforcement mats to overcome this, the reinforcement mats being protected by the cathodic protection.

Columns

The flanking beams would extend across one of the columns supporting the crossbeam, and separate columns (to maintain longitudinal flexibility of the viaduct) were proposed alongside the existing columns to support the flanking beams directly.

Aesthetics

The ends of the flanking beams would be tapered into the existing crossbeam to improve the aesthetics. The discrete location of the crossbeam did not warrant the expense of replacing the beam for aesthetic reasons.

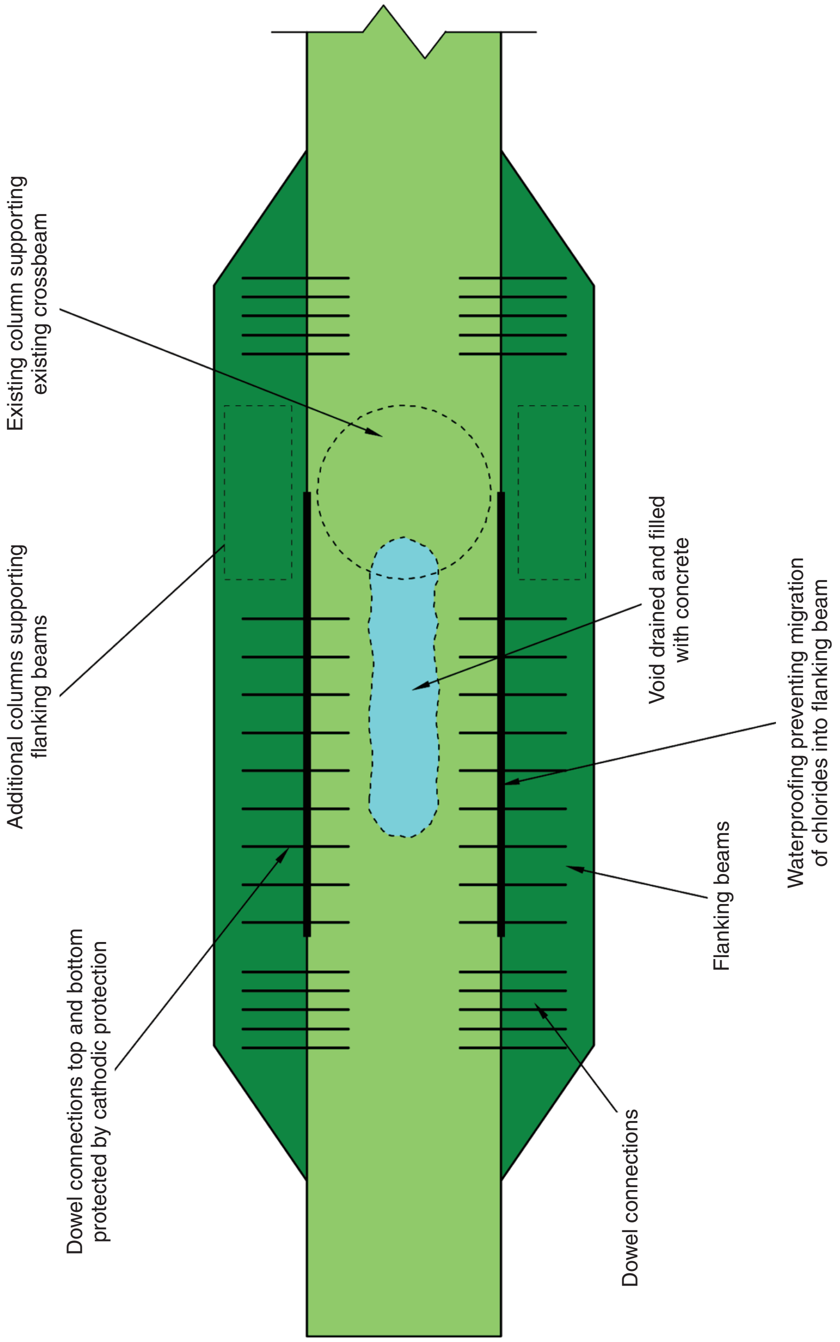

Figure 7.1 – Proposed flanking beams

Case 8 Built-in column repair

Contamination

Circular concrete columns, monolithic with crossbeams and experiencing salt water run-off through leaking deck joints, were found to absorb chlorides at the top of the columns. Presumably run-off from the crossbeam and columns soaked into the ground adjacent to the columns, for the bottoms of the columns were also found to be contaminated with chlorides, possibly by capillary rise from the surrounding ground. Most column delamination occurred at the tops and bottoms of the columns (see Figure 8.1).

Figure 8.1 – Cracks indicating column delamination

Propping

The columns were propped by partially prefabricated steel trestles with HSFG (high strength friction grip) bolted joints, carrying steel trusses or plate girders supporting the crossbeams. (The steelwork was galvanised – see reference to faying surface friction under Case Study 4). However, the concrete columns themselves carried the longitudinal braking forces and distributed temperature forces from the viaduct while propped by the trestles. The main steel bars in the columns thus worked harder in tension under these forces, relieved of dead load compression, while supported by the trestles than they did subject to dead load compression while unpropped. It was thus preferable to repair the columns unpropped, and this also enabled easier access to be obtained to the column for the tasks of repair.

Repair sequence

Calculations were carried out to determine the proportion of the column perimeter which could be broken out to behind the vertical reinforcement such that the remaining encased reinforcement could carry the applied loads. Normally this was between an eighth and a quarter of the perimeter. The circumference at the top and bottom of the column was then broken out in sequence to enable repairs to delamination and to the reinforcement to be carried out (see Figure 8.2). The height of the repairs was determined by the safe height it was considered the repair concrete could be dropped down the repair area without fear of segregation, generally about 1 m. Columns supporting the same or adjacent crossbeams were not repaired simultaneously.

Main bar replacement

Occasionally corrosion of the main bars was so severe that bars had to be replaced. The existing bar was cut out and a new length of bar butt welded in. Strict welding procedures were developed from chemical analysis of the bars and adhered to. Butt welds were never allowed to both ends of an inserted bar since high tensile stresses would develop from the weld shrinkage. However being at the column ends, the bars being replaced were within the starter bar lap length, permitting a free end. Otherwise replacement bars would have had to be lapped in any case within the break out area.

Circular binders

Greater difficulty in fact arose with replacement of the circular binders, which, being closer to the surface of the concrete, suffered much greater section loss. The original circular binders were lapped and fillet welded, so that if the short weld corroded, the binder became ineffective. New lengths had to be inserted and welded at one side of the repair area. A free end was left poking just beyond the other side of the repair area, to which a new length could be welded on if the adjacent area was being repaired. If not, a separate length of binder was welded on each side of the area, and lapped within the area. Failing that, a new length of binder could be welded at both ends, since it was curved, provided there was sufficient play clear of the main bars to accommodate the weld shrinkage.

Cathodic protection

As with the crossbeams, cathodic protection was applied at the tops of the columns so only delaminated areas needed to be repaired. Cathodic protection, however, could not be used reliably below ground, so repairs at the bottom of columns, which usually extended to below ground level could not be so treated. Studies were continuing as to how best to deal with the bottoms of columns. A sound, but expensive, approach would have been to remove all contaminated concrete, and waterproof the column after repair.

Other columns

Some columns were very badly deteriorated and were replaced – see Case Study 14. Other columns supported the crossbeam on bearings and required special treatment to guard against splitting – see Case Study 9. Some columns also suffered from ASR – see Case Study 20.

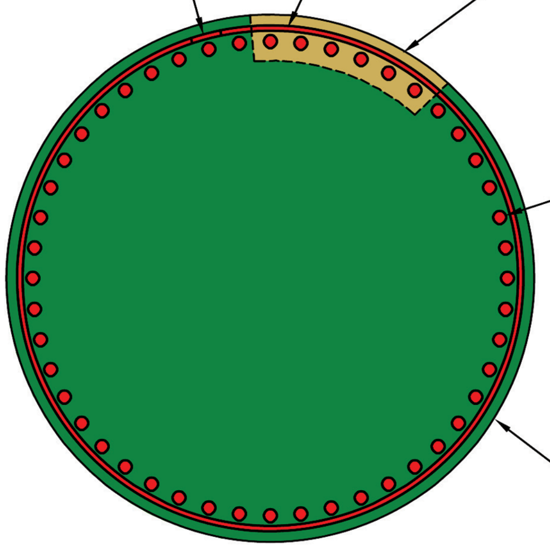

Figure 8.2 – Typical column repair sequence

Case 9 Hinged column repair

Introduction

Circular concrete columns, impregnated with chlorides from leaking bridge joints, were suffering rebar corrosion at the top of the column below rocker bearings supporting the concrete crossbeam above. Unlike built-in columns, the line loading across the top of the column from the rocker bearing could potentially lead to splitting of the column. It was the hoop reinforcement provided to prevent this that was corroding. Some columns were 1500 mm in diameter and of solid concrete. Other columns were 2700 mm in diameter with a solid top supported on an annulus below. Generally there were two layers of hoop steel. On the 1500 mm diameter columns the depth of chloride impregnation indicated it was unlikely that the inner hoops would be corroded. However on the 2700 mm diameter columns the depth of chloride impregnation exceeded 150 mm and there was concern that the inner hoops could be corroded.

Analysis

A strut and tie analysis was carried out to distribute the bearing line loading diagonally downward on to the centroids of the semi-circular segments below, in the case of the 1500 mm columns, and on to the centroids of the two semi-annuli below, in the case of the 2700 mm columns (see Figure 9.1). The horizontal tie forces, half-way down the hoop steel, required to restrain these diagonal struts then provided the forces in the hoop steel. The capacity of the hoop steel allowing for the estimated amount of section loss was found to be inadequate on a number of columns.

Strapping

Temporary steel straps were then bolted around the column tops to compensate for the loss of strength. These were designed in the form of large jubilee clips, the bolts being set in brackets such that at the joint the bolts were recessed into the thickness of the strap to minimise eccentricity and hence minimise bending moment in the strap (see Figure 9.2). The straps were jointed at three locations to facilitate fitting to the column.

Installation

Generally the delaminated concrete was removed along the line of the strap before installation, after which the gap between the strap and the column was filled with grout. Once the grout gained strength, the strap bolts were tightened.

Grouting

On one 2700 mm column, however, the situation was of sufficient concern that further potential weakening of the column by breaking off the delaminated concrete was avoided. The straps were attached around the cracked concrete and the gap filled with grout. The gap behind the delaminated cover concrete was then pressure grouted. Exploratory holes were drilled to check the effectiveness of the pressure grouting. Remaining gaps were then pressure grouted. Further holes indicated almost complete grouting. The strap bolts were then tightened.

Repair

The strap locations were chosen to facilitate repair of the hoop steel, and the straps designed to carry the additional bursting load while the hoop bars between the straps were exposed for repair.

The concrete was then water-jetted out to behind the first layer of hoop steel, and replacement lengths of hoops welded in where required, before reconcreting.

Additional straps were then attached around the repaired areas, and the first straps removed so that the areas behind them could be repaired, and the hoop bars replaced as required.

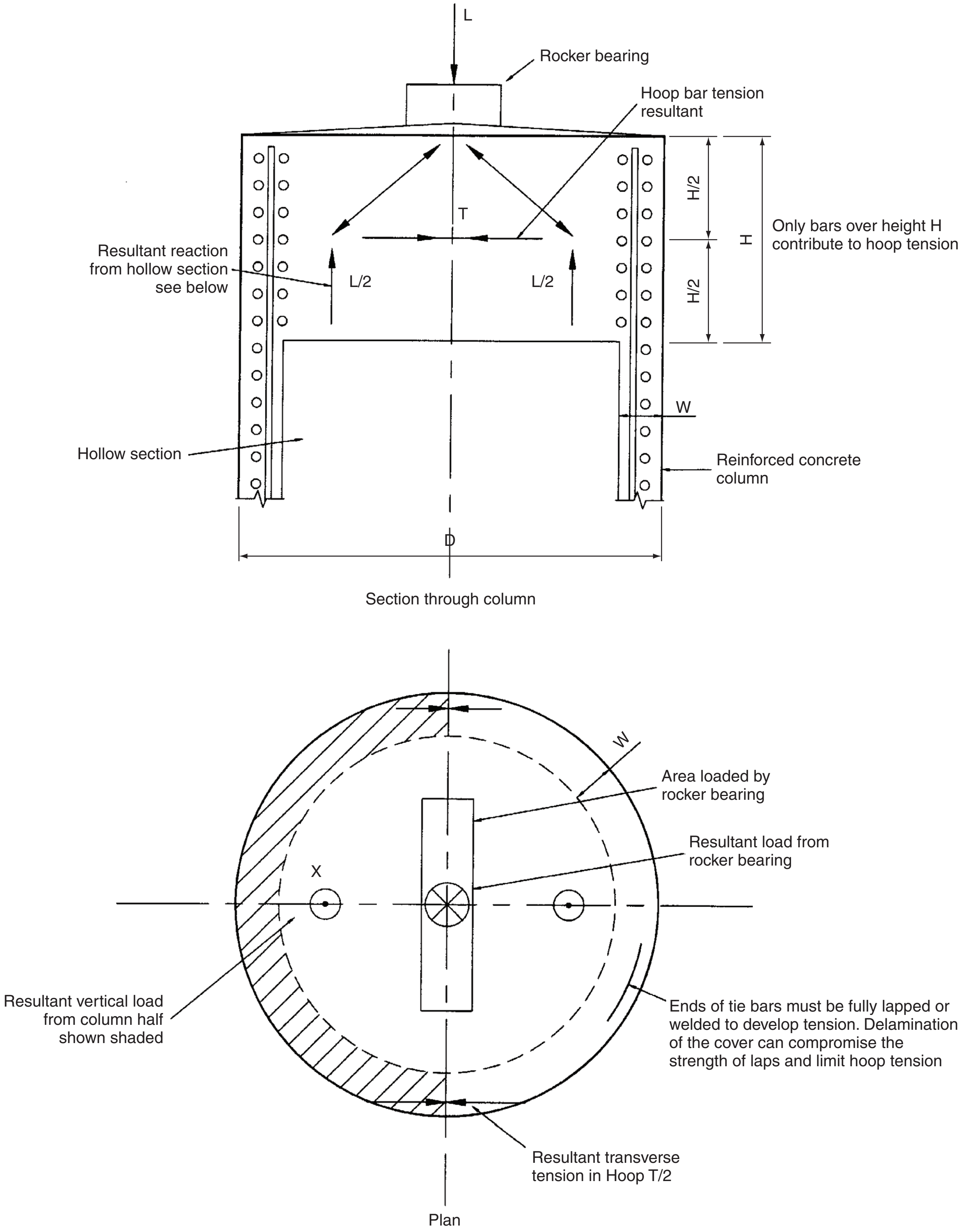

Figure 9.1 – Strut and tie analysis

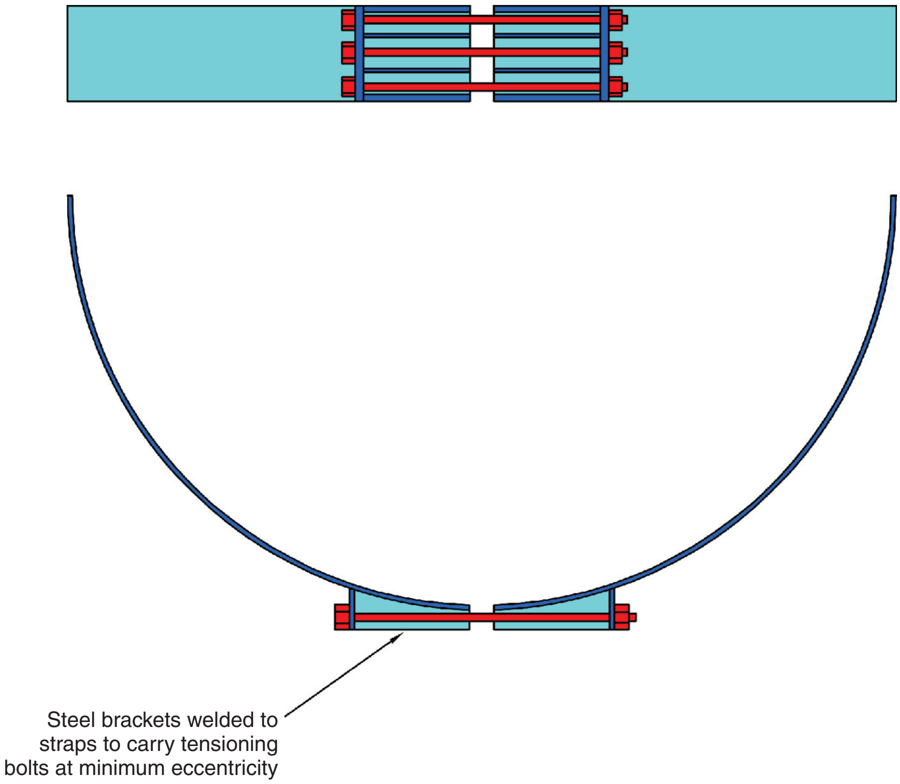

Figure 9.2 – Steel column straps

On the severely affected 2700 mm diameter columns the inner hoop reinforcement was also exposed for inspection. Plans had been made to replace this if required, but in the event this was not necessary.

Cathodic protection

Cathodic protection was then applied to the column tops, it having been established that sufficient penetration should be achieved to prevent further corrosion of the inner hoop reinforcement, to which depth the chloride contamination had at some locations penetrated.

Case 10 Deteriorated crossbeam

Introduction

A crossbeam was exhibiting serious delamination following chloride attack from road salts leaking from above and a full examination and testing regime was put in motion.

Condition

The delamination on the top of the beam was so severe that fingers could be inserted into the cracks along the top edge of the beam. Most of the surfaces of the crossbeam were delaminated.

Delaminated areas were lifted off and exposed corrosion so severe that link bars were corroded right through, and some main bars had lost up to 40% of their cross sectional area. Reinforcement was slightly corroded at a depth of 100 mm from the surface (see Figures 10.1–10.3).

Figure 10.1 – Torsion bar corroded right through

Figure 10.2 – Severe main bar corrosion

Figure 10.3 – Wide delamination crack in crossbeam top

Propping

The beam was propped as a matter of urgency using steel trusses supported on steel trestles erected around the columns.

Properties

The beam surface looked porous (see Figure 10.4) and the porosity of the concrete was found to be moderately high. The strength of the 20 year-old beam had only just reached 21 N/mm^2 against a specified strength of 29 N/mm^2. The cement content of the concrete was about 12%, only two thirds of the content that would have been expected. The weak beam had been much more absorbtive of chlorides than full strength beams subject to similar run-off of road salts, and chloride contents by weight of cement were up to 60 times the permissible limit of 0.2%. Even at 190 mm depth the chloride limit was exceeded. Sulphate contents were satisfactory and although the aggregates were reactive, ASR was considered to be level 2 and no gel was found in thin sections.

Figure 10.4 – Poor quality concrete

Bearing stability

Bearing holding down bolts were also badly corroded and there was concern that before long the bearings could become loose or unstable with delamination progressing below the bearing plinths. The steel supports did nothing to overcome such problems.

Shear links

The depth of chloride attack was such that the shear links could fail completely, and with only some 20–30% of the live load carried on the steel propping there was even a question of the possibility of the crossbeam failing between the jacking points.

Columns

The concrete columns were of concrete of specified quality, although the strength was slightly on the low side.

Solution

The entire crossbeam was replaced as repair would have entailed difficult replacement of main steel bars, and cathodic protection could not be guaranteed to penetrate to the depth needed to safeguard the shear reinforcement.

In any case it may not have been practicable to carry out sufficient repairs to the shear reinforcement.

The replacement procedure was similar to that described in Case Study 2 (see Figure 10.5).

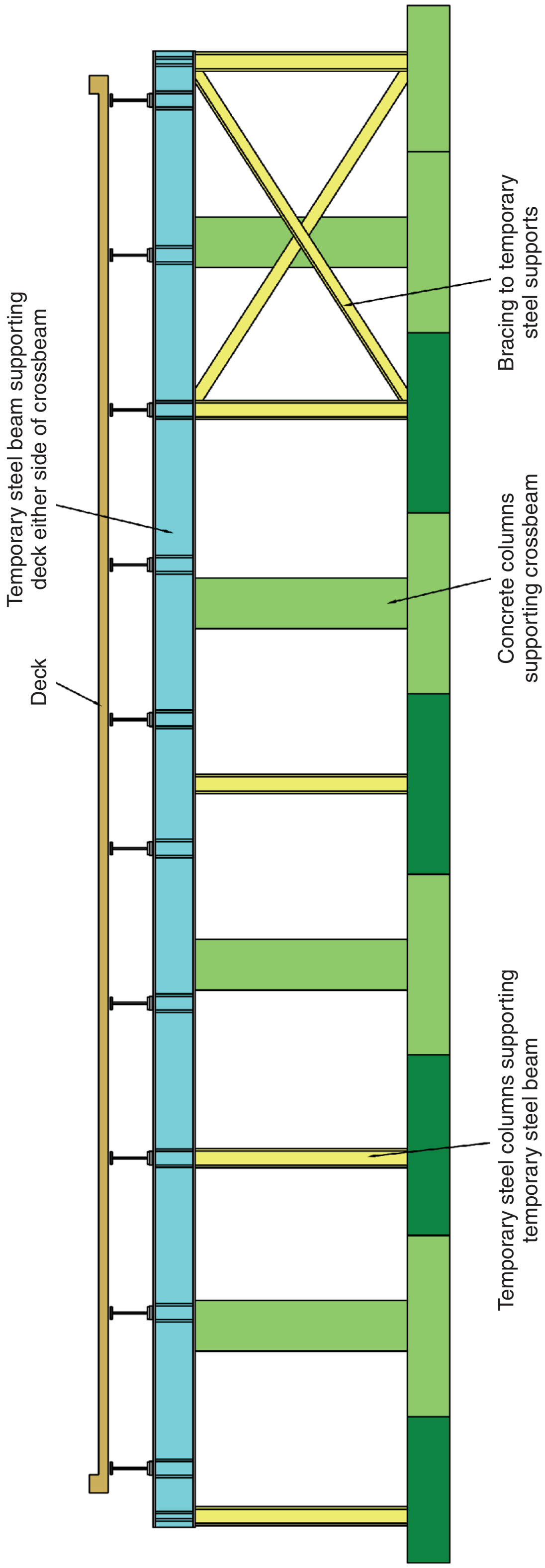

Figure 10.5 – Propping for replacement

Case 11 Deteriorated columns

Introduction

Columns in the vicinity of the deteriorated crossbeam described in Case Study 10 were exhibiting more delamination and spalling than were others. Investigations were carried out and it was found that the cement content was also low, although not as low as in the deteriorated crossbeam.

Analysis

Due to high chloride impregnation of the weak concrete, the links in the lap zone of the column starter bars were severely corroded and much of the lap zone was spalled or delaminated. There was concern that the bond strength of the lap was insufficiently effective to carry the moments in the columns, particularly due to braking forces. The ability of the columns to fully withstand the vertical loading, combined with compression due to the bending moments, was also calculated to be in doubt as a result of the weak concrete.

Temporary propping

The decision was taken to prop the columns without delay. Trestle steelwork for supporting trusses was available from the crossbeam repair programme, and this was used at the critical columns. At the top of the trestles were steel bearing beams upon which the bearings carrying the trusses were normally installed. The trestles were turned through 90° in plan for use in propping the columns, so that the bearing beams passed under the crossbeam above, and could be used to carry the jacks to apply some preload in the trestles (see Figure 11.1). The integrity of the columns was thereby safeguarded until a procedure could be devised for their replacement.

Replacement propping

When a project was later devised to replace the deteriorated crossbeam, replacement of the columns was included. The propping described in Case Study 14 was to be increased in height and reused, it having being designed since the columns in question were propped, as adequate access for the work was not possible within the temporary trestle props.

Switching propping

In order to switch the propping, temporary repairs were effected to the lap zone at the bottom of each column, and circular straps installed to enable adequate bond strength to be developed, as described in Case Study 9. With a load restriction in place, the trestle supports were removed and the propping for replacement installed.

Replacement

The procedure for replacement was then that described under Case Study 14.

Case 12 Sagging weakness in crossbeam

The problem

A weakness was found in a concrete crossbeam at mid-span during assessment. It appeared that the crossbeam had been detailed using 10 main bars in the soffit instead of the intended 20. The motorway carried by the crossbeam was offset along the beam so that by closing the hard shoulder as a precautionary measure, the overstress was reduced from 16 to 13%. The soffit was cracked at the weak point and the crack was monitored to ensure it did not open further until temporary propping could be erected.

Condition

Although the crossbeam had suffered chloride run-down from a leaking deck joint above, the chloride content of the concrete and the half-cell readings at the weak point and at the adjacent lap zone were low, and there was no delamination at the lap zone. The deck joint had been replaced and a gutter installed under it to catch any run-off should the joint leak again. Salt was by then only used in exceptional circumstances, de-icing being normally carried out by the use of urea. The risk of loss of section of the ten bars or of loss of bond strength in their anchorage zone was therefore low. Nevertheless half-cell readings and hammer tapping surveys were regularly carried out as a precaution.

Additional reinforcement

Investigations were carried out in order to evaluate how to strengthen the crossbeam. The first proposal was to add further bars to the soffit. This could have been carried out fairly simply during soffit repairs, although the bars would have to avoid the jack locations for the temporary propping. However there was difficulty in how to install the additional bars to carry the longitudinal shear forces between the additional reinforcement and the existing beam and these would either have to be drilled into the soffit, or welded to the existing links, neither of which provided a practicable solution due to congestion of reinforcement.

Carbon fibre reinforced plastic (FRP)

Strengthening with carbon fibre was then considered. However, there were two main difficulties. The beam was deep and stiff and the E-value of carbon fibre much lower than that of the existing steel bars. To generate a reasonable proportion of the strength of the carbon fibre, the strain in the carbon fibre would mean that the existing bars would strain beyond their yield point. Thus it was impracticable to share the load between the carbon fibre and the normal reinforcement.

Thus either all the load would have to be carried on the carbon fibre, or sufficient carbon fibre provided so that it would provide adequate strength at a strain below the yield of the steel reinforcement. Either way a significant number of layers of carbon fibre would have to be provided.

This led to the second difficulty. In the zones where the carbon fibre would be anchored, there were steel rebars at close centres. The peeling force of these layers of carbon fibre could have delaminated the concrete cover to which the carbon fibre was attached, and which had little strength since it relied on concrete in tension in the narrow zones between the bars. Common practice to avoid peeling was to bolt the end of the carbon fibre into the concrete, but drilling holes for bolts between the close spaced rebars would have been difficult in practice. Furthermore adhesion of the carbon fibre to the cover concrete was still essential to develop adequate anchorage, and any corrosion of the steel reinforcement leading to delamination would have negated this strength. Risk of sudden failure was thus too great for the carbon fibre option to be considered further.

Case 13 Hogging weakness in crossbeam

Introduction

During assessment two crossbeams carrying the motorway over a dual carriageway were found to have a weakness in hogging over their columns. An inadequate number of bars formed the adjacent lap length. The beams were found to have vertical cracks at the weak point. As emergency propping would have closed the dual carriageway, the hard shoulders were closed to relieve the overstress.

Strengthening options

Various proposals were considered for the strengthening, such as adding additional bars to the top of the beam, and applying carbon fibre. Similar difficulties were encountered as described in Case Study 12. However on the top of the crossbeam there were further problems in that panel walls built above the middle of the crossbeam precluded use of some of the space on top of the beam, and the bearing plinths masked too much of the remainder.

Nibs

The solution adopted was to cast reinforced concrete nibs on the top corners of the crossbeams, similar to those provided in Case Study 1, while the crossbeams were propped for repair of chloride contamination.

The problem

The preferred means of carrying out the works across the dual carriageway was to divert the road around the ends of the crossbeams, which could then be propped. However, the adjacent landowner wanted a great deal of money for the use of his land. What was needed was an alternative solution so that the landowner could be offered a reasonable sum on a ‘take it or leave it’ basis.

Cranked girders

The answer was a pair of cranked girders with which to prop the crossbeam (see Figure 13.1). Part of the length of the girders were to be erected below the level of the crossbeam, enabling that length of crossbeam to be repaired in sequence all round, above the girder. At the end of that length, the girders quickly ‘cranked up’ to lie either side of the crossbeam until they reached the next support, and beyond. The traffic on the dual carriageway was diverted under the higher lengths of the girder.

Traffic management

After this first stage of repairs was completed the short cranked length of girder was disconnected and reversed. The girder was then to be re-erected with the lower section at the higher level and the higher section at the lower level. The traffic was diverted to run under the higher section of girder, and the crossbeam repaired above the lower girder length.

Solution

This proposal appeared both practicable and fairly economical, and the land owner was persuaded to lend his land at a sensible price, enabling the dual carriageway to be fully diverted and the strengthening and repairs to be carried out using conventional girders for the propping. It was therefore possible to carry out the work more efficiently than by the use of the cranked girder.

Case 14 Column replacement

Introduction

Circular concrete columns supporting concrete crossbeams were badly deteriorated due to chloride run-down causing corrosion and delamination. A number, 1200 mm in diameter, and some 4–5 m in height were considered sufficiently badly deteriorated to warrant replacement rather than repair.

Propping

Trestles had been designed to support trusses for crossbeam repair, but the diagonal bracing over the full height on all four sides precluded sufficient access for both demolition and reconstruction. A new design of propping with universal columns at each corner and horizontal bracing at top and bottom was designed (see Figure 14.1) This had sufficient stiffness without intermediate bracing, and yet adequate flexibility to sway with the viaduct under braking and temperature movements so as not to pick up excessive transverse forces. The crossbeam and its remaining columns were analysed with the propping in place both with and without the replacement column to ensure that no element became overstressed.

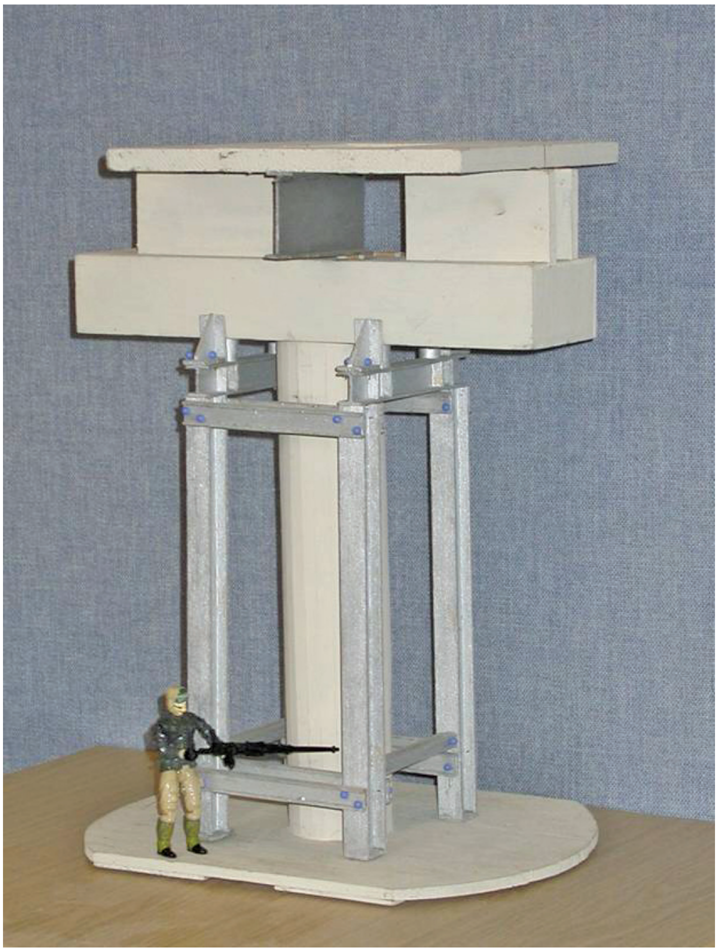

Figure 14.1 – Support for replacement of 1200 mm diameter column

Brackets

Once erected, the column dead load was to be jacked into the propping. Brackets were attached to the sides of the crossbeam against which the propping was to be braced to provide transverse stability.

Model

There was concern that the propping might be too close to the column to allow the starter bars from the foundations and those projecting into the crossbeam to be water-jetted out. At design stage a balsa wood model was built at 1 in 20 scale of the column, crossbeam, propping and its plinths and a 1.8 m tall model at 1 in 20 scale positioned with a scale water-jetting lance aiming it at the column adjacent to the plinths. It was seen that the rectangular plinths would not allow the column to be water-jetted, and they were turned at an angle tangential to the column. By the same method the propping was seen to be sufficiently clear of the column. This approach worked very well and the water jetting proceeded smoothly in practice (see Figure 14.1).

Removal

It was anticipated that the column might be severed between the top and bottom starter bars by diamond band saw, and lifted out in a single piece. Calculations were carried out to check that the column could safely be swung clear of the propping. Allowance was made for the suspension chain on one side of the column snapping. The angular momentum of the column swinging on the single chain was calculated and found to be sufficient to badly damage the propping. In the event the whole column was demolished by water-jetting, which was carried out much more quickly than expected.

Reconcreting

Precautions were taken to ensure that the aggregate was sufficiently small to enable the concrete to flow around the congested starter bars, that the concrete was not poured from an excessive height, and that it was adequately vibrated.

Creep and shrinkage

Care had to be taken to design the column to minimise creep and shrinkage, since the column was built into the crossbeam. Sufficient reinforcement was provided to control these. Initial shrinkage was allowed to take place before the closing section of the column was cast. Finally a shallow height of grout was poured in from one side, such that it gradually filled up to an inclined soffit, thereby expelling air ahead of it.

Dejacking

Lastly the load from the propping was dejacked into the column. The column had been cast up to the crossbeam at a level which allowed for column compression, and eventually some creep, once the load was applied.

Problem

A problem did occur in that by the time the concrete had reached sufficient strength, it had shrunk down, increasing the load on the jacks and it was only just possible to release the locking rings. Thereafter locking rings were eased daily to allow for shrinkage forces and movements.

Wider columns

Supports were also designed on the same principle for the refurbishment of badly deteriorated 1500 mm diameter columns. Due to the much heavier loads adequate stiffness was provided by tubular steel columns erected directly under the crossbeam. In the event the columns were repaired rather than replaced (see Figure 14.2).

Figure 14.2 – Support for repair of 1500 mm diameter column

Case 15 Downstand replacement

Introduction

A crossbeam supported a steel concrete composite deck span and a solid concrete slab deck span, the latter having a reinforced concrete downstand to make up to the depth of the adjacent composite deck. The crossbeam was impregnated with chlorides from the leaking deck joint above and it was proposed to replace the delaminated concrete and apply cathodic protection. However, the reinforced concrete downstand, in covering half the width of the top of the crossbeam, would preclude access and application of the cathodic protection. The decision was taken to remove the downstand and replace the simple downstand concrete line rocker with discrete bearings (see Figure 15.1).

End anchorage

The major difficulty with removing the downstand was that the longitudinal reinforcement at deck soffit level was not sufficiently well anchored at the deck end, relying on the reinforcement in the downstand to provide that. Bars anchored by small plates retained by nuts on a threaded end were therefore drilled and grouted into the end of the deck to overlap and anchor the existing reinforcement. The technique used was that the bar was rotated in the hole, thereby mixing the two part epoxy enabling it to harden. A selection of the bars was tested to ensure satisfactory application.

Replacement procedure

Once the epoxy grout had hardened, the downstand was removed by water-jetting over lengths just sufficient to insert the new bearing. Prior to bearing installation, any delamination on the top of the crossbeam was removed and replaced with repair concrete, the crossbeam having been temporarily propped. There was sufficient height available to drill in anchor bars for the bearing downstand and holding down bolts to fix the bearing (see Figures 15.2 and 15.3). This process was repeated until all the bearings were installed, after which the remainder of the downstand between the bearings was removed, and the repair of the crossbeam completed.

Articulation

The crossbeam supported the decks at a skew angle and the ends of the steel girders supporting the composite deck did not have bearing stiffeners (see Figures 15.4 and 15.5). A number of other crossbeams supported composite decks at skew angles, and calculations showed that bearing stiffeners needed to be added at all these crossbeams. The transverse forces at the deck ends were carried by reinforced concrete panel walls built on to the crossbeams at deck ends which were fixed longitudinally and by shear keys between the adjacent deck ends at those which were free to move longitudinally. At the free ends, the deck edge was supported by reinforced concrete stub columns, free to rotate about both longitudinal and transverse axes (see Figure 15.6).

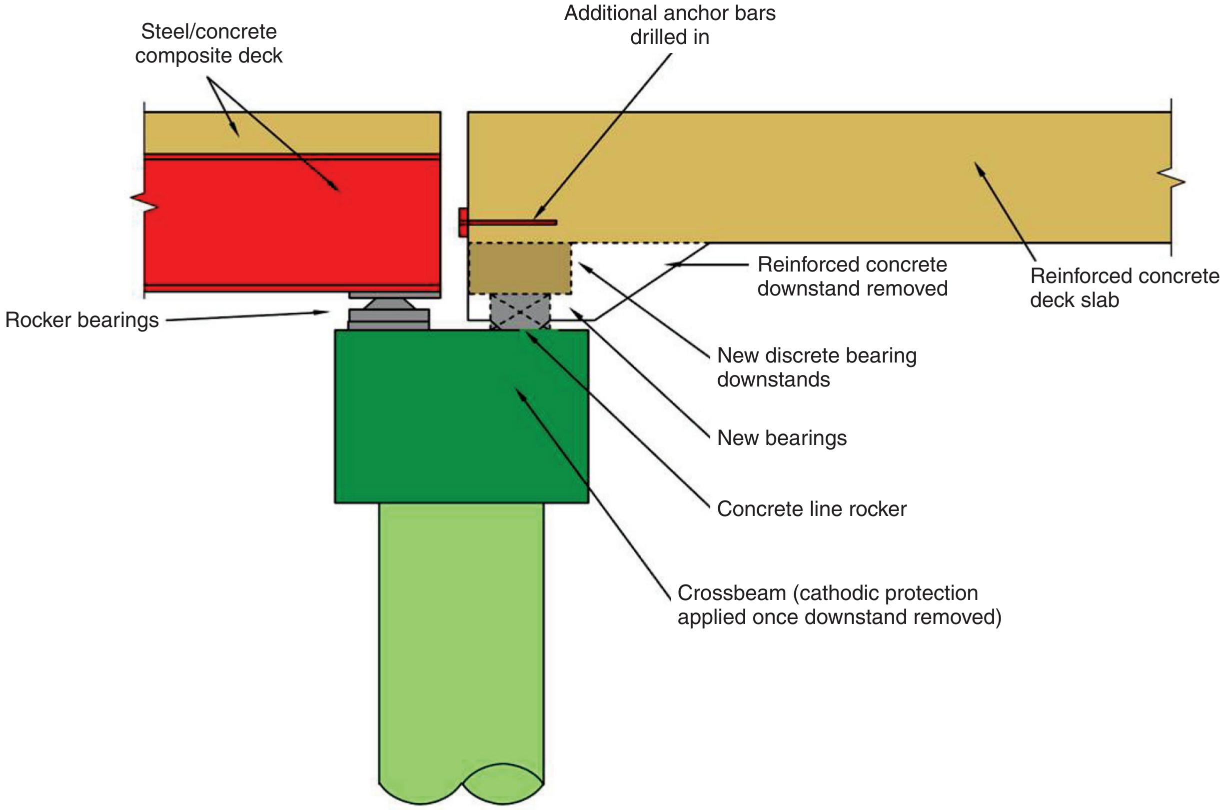

Figure 15.1 – Downstand replacement

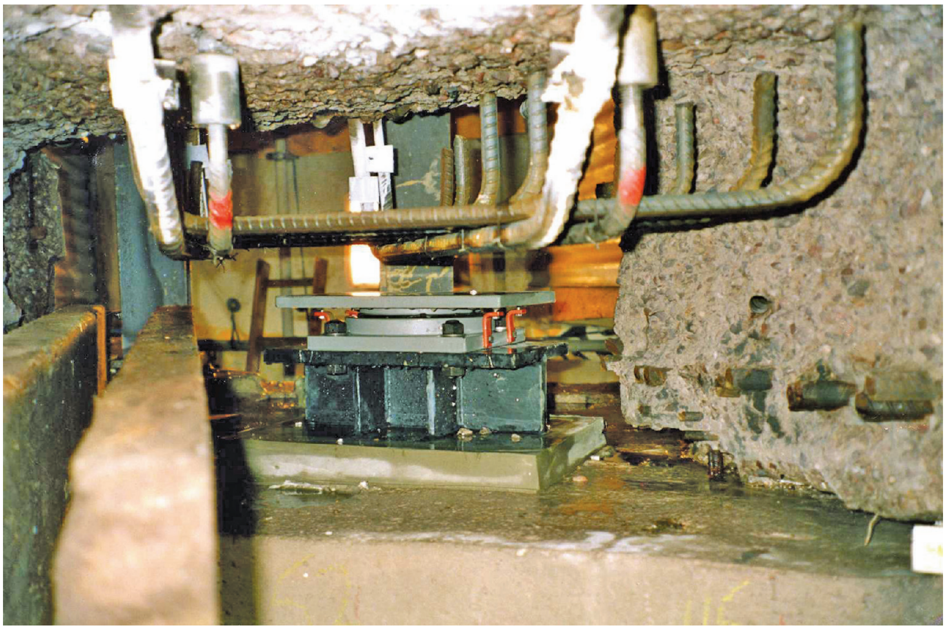

Figure 15.2 – Reinforcement for bearing downstand

Figure 15.3 – Reinforcement for bearing downstand

Figure 15.4 – Viaduct supported on skewed crossbeams

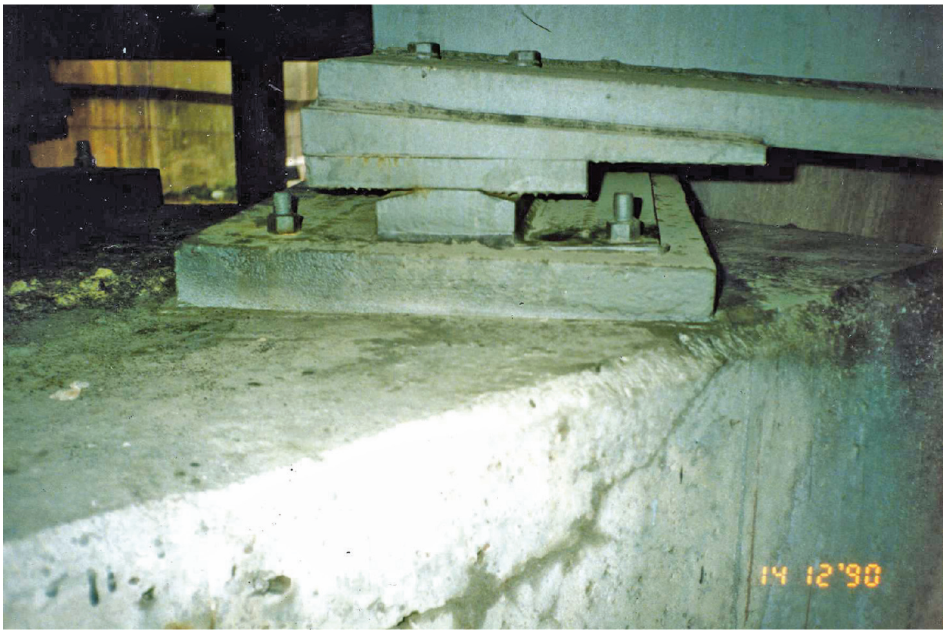

Figure 15.5 – Steel rocker bearing. No bearing stiffeners

Figure 15.6 – Stub column

Stub columns

The stub columns were severely corroded, the perimeter often being fully delaminated due to corrosion of the helical reinforcement, and the dowels into the deck sometimes being corroded right through (see Figure 15.7). On one crossbeam the stub columns had been temporarily supported on either side by hardwood props, with timber walings front and back. Although on a skewed support, the steel deck girders were supported on simple steel line rocker bearings, these being required to support the web in the absence of bearing stiffeners. However, calculations showed that this arrangement would become overstressed with the addition of bearing stiffeners, and the rocker bearings were therefore to be replaced with spherical bearings. These in turn required the bottom flanges of the deck girders to be braced (see Figure 15.8).

Figure 15.7 – Discarded deteriorated stub columns

Figure 15.8 – Bracing to bottom flange. (Also trimmer beam)

Stub column replacement

The reinforced concrete stub columns were then suitably replaced by a steel trimmer beam supporting the end of the deck and braced down to the new bearing stiffeners at the bottom flange (see Figure 15.9). Temporary steel props were installed and temporarily fixed to the deck soffit to enable the stub columns to be removed before offering up the steel trimmer beam and bolting it to the newly welded bearing stiffeners (see Figure 15.10).

Figure 15.9 – Trimmer beam

Figure 15.10 – Temporary propping to deck

Case 16 Deck repair

Introduction

Dual three lane plus hard shoulder motorway steel concrete composite deck slabs suffered chloride attack from road salts and were to be repaired during resurfacing. The deck ends were generally affected and the deck joints had leaked. To keep repairs to an economical level and traffic disruption to a minimum, delamination surveys were carried out at the deck ends once the surfacing and waterproofing were removed. Half cell readings were taken around the delaminated areas, and those areas of concrete with numerically high half-cell readings were removed in addition to the delaminated areas.

Fatique

Where required new lengths of rebar were lapped or welded in (see Figure 16.1). However, because the deck end was supported on panel walls built on the crossbeam below, and therefore subjected to high fatigue cantilever stresses under the action of repetitive wheel loads, welding of the longitudinal rebars was disallowed within some 700 mm of the deck end. Bars had to be lapped in, or the concrete broken out further back into the span to permit welding.

Figure 16.1 – Deck rebar pitting corrosion

Repair sequence

Although unlikely, a repair sequence was developed to cater for the possible need to repair the deck slabs remote from the deck ends (see Figure 16.2). The maximum permissible repair area was calculated, allowing for the transverse loading to be carried around either end of the repair. This was based on the maximum load of 100 tonnes permitted during operation of the contraflow of three lanes of traffic in each direction (without hard shoulder) permitting two lanes to be free for maintenance, one being under repair and the other available for construction traffic. Plans were then prepared of the standard and non-standard deck types, showing the sequence to be followed such that, if required, any part of any deck could be repaired allowing also for the traffic management programme. The structural calculations had to allow also for the loss longitudinal shear restraint where repairs crossed the deck beam shear studs.

Non-standard decks

The non-standard decks comprised skewed and tapered steel/concrete composite decks, and trapezoidally shaped concrete slab decks linking the standard composite decks with skewed viaduct crossings. Some slab decks comprised transverse downstand beams supported on columns, a downstand beam also acting as a trimmer beam at the skewed end. The tops of the downstand beams could only be repaired in a half-width at a time, and over short transverse lengths dictated by the traffic management sequence. Where the skewed trimmer beam met the transverse beams, the permitted repair areas were even more complex. Other slab decks were of uniform depth supported on a matrix of columns. The repair areas at the columns were sufficiently small to ensure that the punching shear strength was not compromised.

Traffic vibration

There was concern that traffic vibration would adversely affect the setting of the repair concrete, but a paper from the United States by Manning had concluded that such vibration was generally beneficial, and a departure from standard to permit repairs adjacent to live traffic was obtained on that basis.

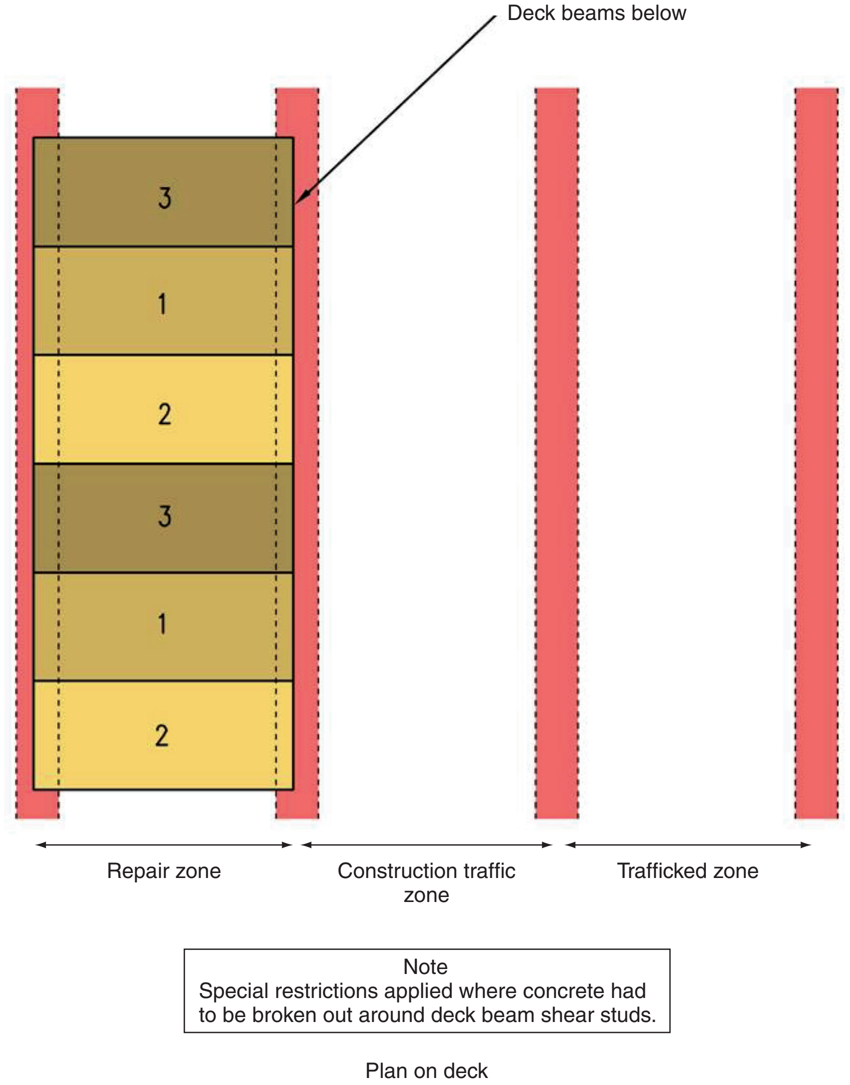

Figure 16.2 – Typical deck repair sequence

Case 17 Column strengthening for impact

Introduction

There was a concern that columns of motorway viaducts would not be strong enough to sustain impact damage from errant vehicles and calculations were carried out for all columns in vulnerable positions. The columns were normally 1200 mm or 1500 mm in diameter and these were generally sufficiently robust.

Three level interchange

However, there was a problem where one viaduct crossed another at a skew angle, and a vehicle accidentally leaving the lower deck could impact a column supporting the upper deck. Once allowance was made however, for the retarding action of the vehicle penetrating the precast concrete parapet, the situation was found to be much less critical.

Strengthening

There were nevertheless overbridges across the motorway supported by slender raking columns and these could only sustain errant vehicle impact loads of one eighth that recommended. The proposed solution drawn up was to infill the gaps between the raking columns with a reinforced concrete slab, providing greatly enhanced restraint to impact longitudinal to the motorway (see Figure 17.1).

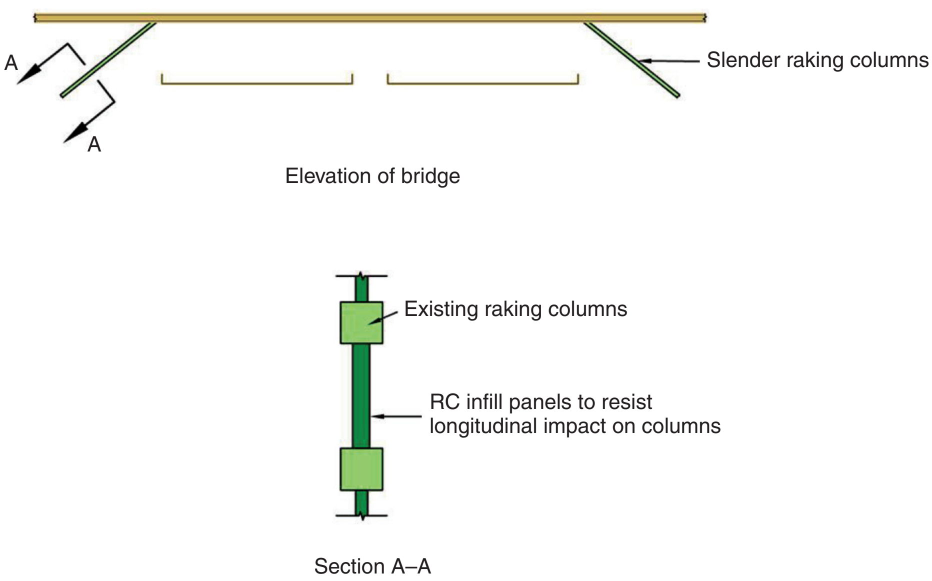

Figure 17.1 – Column infill strengthening

Case 18 Opening deck joint

Introduction

Within a line of motorway viaduct simply supported spans, an expansion joint at a skewed railway crossing was seen to be opening up in low temperatures. After monitoring the movement for some time, an investigation was carried out to find out if there was a structural problem.

Articulation

The ends of the steel concrete composite decks were supported on reinforced concrete panel walls built on top of the crossbeams which flexed under expansion movements. The steel beams were supported on simple sliding rocker bearings whereby the bottom flange of the beam slid on the top plate of the rocker, the sliding surfaces having originally been lubricated with graphite. After 20 years in service the friction of the sliding surface had increased, so that sliding occurred only between several spans at increased temperature changes.

Structure at joint

The series of composite spans was connected to a trapeziodally shaped reinforced concrete slab deck supported on slender columns which formed the approach to the railway crossing. The trapezoidal deck abutted, but was not connected to, the skewed supported railway crossing deck. A similar joint was installed at the far side of the crossing, but the continuous length of viaduct beyond was much shorter.

Analysis

A non-linear analysis under temperature changes was carried out of the length of viaduct extending from the joint, modelling the decks, columns, panel walls and sliding bearing friction in two dimensions. A typical range of friction values was known from friction tests carried out earlier to ascertain whether the bearings could be left in service. As the temperature was decreased, the bearings subjected to the greatest friction forces were allowed to slip in turn, and the movement at the joint recorded. This was then calibrated against the monitored movements.

Result

It was found that given the increased friction on the bearings, reducing the temperature swayed the columns approaching the joint, forcing the joint to open up. However, the forces involved were not excessive either on the panel walls or on the columns, and so, provided the bearing frictions did not further significantly deteriorate, there was no structural danger.

Expansion joint

However, the existing expansion joint had not been designed to cater for these movements, nor for the shearing movement on the highly skewed joint. A new joint was therefore selected and installed to cope with these effects.

Case 19 Dropped deck bearing

Introduction

The friction values of simple sliding rocker bearings supporting steel concrete composite viaduct decks had increased over time, increasing the transverse forces on the bearings, and contributing to deterioration of the bearing plinths. A programme was undertaken of regrouting the bearing plinths. This was achieved by inserting low-height jacks under the steel beam flange in front of the bearing.

The problem

Damage was later noticed to the deck slab soffit either side of one of the deck beams. Investigations showed that the beam had dropped during the bearing plinth replacement, pulling the lower mat of deck reinforcement with it, and delaminating and spalling the soffit cover. Corrosion had set in (see Figures 19.1 and 19.2).

Jacking

The beam was jacked up, but aggregate particles had filled the gap above the beam around the shear studs and it was not practicable to restore the beam to its intended level.

Repair

Proposals were drawn up to remove the delaminated concrete, pressure grout the gap above the beam, and spray concrete the damaged slab. However, the viaducts were due to be resurfaced and the slab was monitored until the full depth of the deck could be reconcreted from above.

Figure 19.1 – Deck soffit corrosion

Case 20 Alkali Silica reaction (ASR)

Introduction

For a number of years vertical cracks were observed and monitored on the centre line of a number of beam ends on the Midland links motorway viaducts. Investigations for ASR indicated that while there was some evidence of ASR the cause of the initial cracking was unlikely to be associated with ASR.

Thereafter the cracks opened and developed into associated horizontal cracks, diagonal cracks along the side of the beams, and multiple cracking and crazing on the beam ends. Typical crack patterns are shown in Figure 20.1.

Inspection

The vertical cracks on the beam centreline varied in width from very fine to 5 mm. On one beam they continued over 50% of the length of the top of the crossbeam and on some beams there was more than one vertical crack. Vertical cracking also occurred through the cathodic protection coating of beams previously repaired. The cracks moved about 0.6 mm in response to expansion and contraction of the simply supported decks.

Horizontal cracks occurred on the beam end, varying from very fine to 1.5 mm. In some instances these propagated into diagonal cracks along the side of the crossbeam, up to 2 mm in width at the crossbeam end, tapering to fine some 1.5 m from the beam end.

The diagonal cracks were located in various positions on different crossbeams. In relation to the imaginary diagonal compressive strut supporting the bearing, the diagonal cracks ran along the strut, parallel to and below the strut, well below the strut and across the strut.

Multiple vertical and horizontal cracks occurred on several beam ends, and extensive crazing also occurred.

Causes

Earlier petrographic inspections generally indicated Level 2 ASR with a few results indicating Level 3, which suggested that ASR could not explain the degree of macro-cracking found. It was not therefore clear how the cracks first originated. The original cause may have been shrinkage due to the beam end drying out faster than the core of the beam. (Many of the affected beam ends face south across the adjacent canal). The formation of such cracks would have let in water due to the exposure to the weather of the beam end beyond the deck edge, thereby encouraging more active ASR.

ASR

A later petrographic inspection revealed an ASR Level of 4 out of 10. The concrete constituents were potentially reactive and the cement content in the affected beams was higher than average, which would encourage ASR.

Summing crack widths is an accepted indication of current expansion due to ASR. Earlier this was determined as 0.13 mm/m after allowing for drying shrinkage, which would not indicate an ASR problem. Later the widths indicated 1–3 mm/m transversely and 1 mm/m vertically. If due to ASR alone, these would indicate a considerable degree of severity, but as it was not known to what extent these figures were affected by shrinkage and by deck contraction, no firm conclusions could be drawn from them with regard to ASR severity.

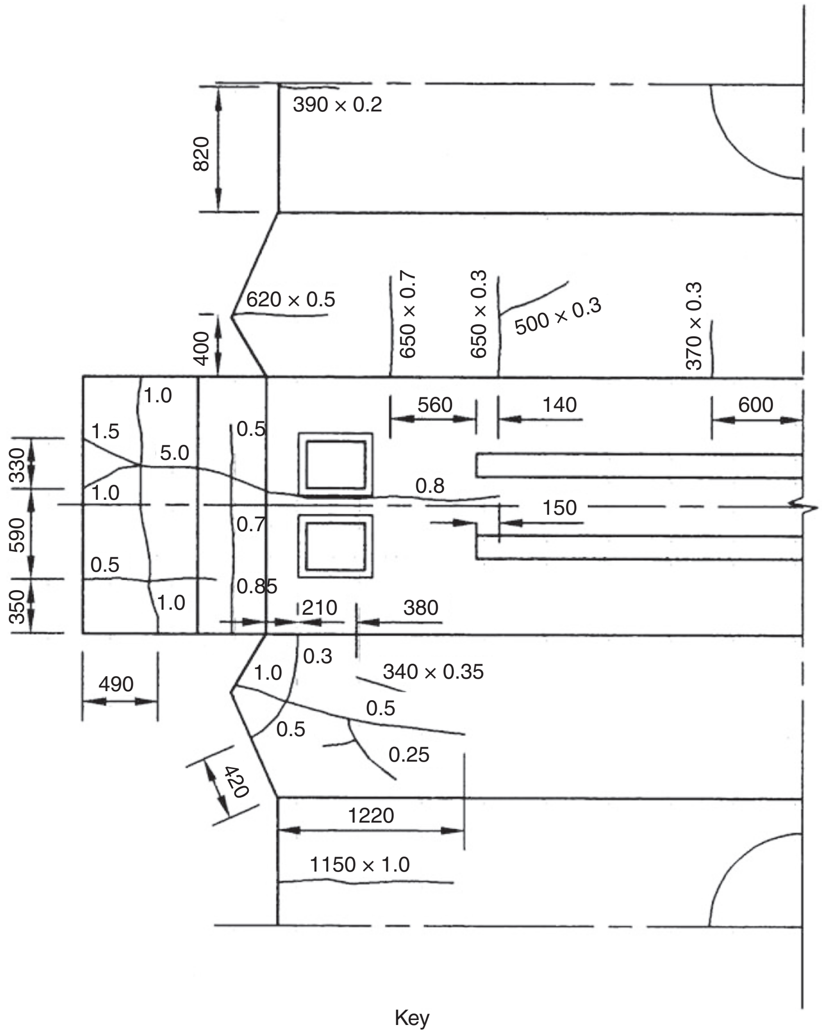

Figure 20.1 – Typical cracking on beam end

Accelerated expansion tests previously carried out, indicated that maximum free expansions could reach 0.71 mm/m. While that did not greatly exceed the threshold of 0.6 mm/m below which ASR is not a problem, it was significant given

- the exterior position open to the weather which was a wet environment;
- the lack of an adequate three dimensional cage of reinforcement for containing the expansion;
- the fact that the diagonal cracking could undermine the deck beam bearings supporting the hard shoulder.

Under the above circumstances the IStructE publication *Structural Effects of Alkali-Silica Reaction – Technical Guidance on the Appraisal of Existing Structures* suggested that the severity of ASR could be A – the most severe level.

Consequences

Whether the principal cause of cracking was shrinkage or ASR, there was insufficient reinforcement in the beam to control it.

The situation was exacerbated by the effects of deck contraction. Calculations showed that the 12 mm diameter mild steel bars at 300 centres which passed round the full perimeter of the beam cross-section would yield under extreme temperature contraction movements of the decks if the bearing friction exceeded 0.32. However bearing frictions of up to unity were quite possible due to corrosion of the sliding surface between the deck beams and the bearing plates. With vertical cracks up to 5 mm, it was apparent that these bars had yielded. Repeated temperature movements could cause the bars to fail in fatigue.

Integrity

Where the vertical cracking exceeded 2 mm, there would be a loss of aggregate interlock, and the two halves of the beam would act independently, allowing one half of the beam to become overloaded under eccentric loading.

The structural effects of ASR can be limited, because the section core expands due to the ASR, while in the surface layer the degree of reactivity may be reduced due to leaching of alkalis by water or by a reduction in alkalinity due to precipitation of sodium and potassium carbonates. This results in much less expansion of the cover zone, limiting macro-cracking to the cover zone. For free expansions of up to 1 mm/m the compressive strength of the core can be weakened by up to 15% and the tensile strength by up to 25% due to micro-cracking caused by gel formation.

A restriction in loading on the hard shoulder to 25 units of HB was already in place due to delamination of the cantilevers. The beam ends exhibiting cracking were not delaminated, however, and so the existing load restriction was able to compensate for loss of strength due to the cracking or ASR.

Action

The hard shoulder was closed where the vertical crack was over 2 mm in width. Beam ends with vertical cracks 1 mm or greater in width were strapped transversely with steel channels braced with 25 mm diameter Macalloy bars either side of the deck edge beam, designed to withstand deck contraction forces transmitted to the bearings, assuming a bearing friction co-efficient of unity, to prevent the vertical cracks being pulled open (see Figure 20.2).

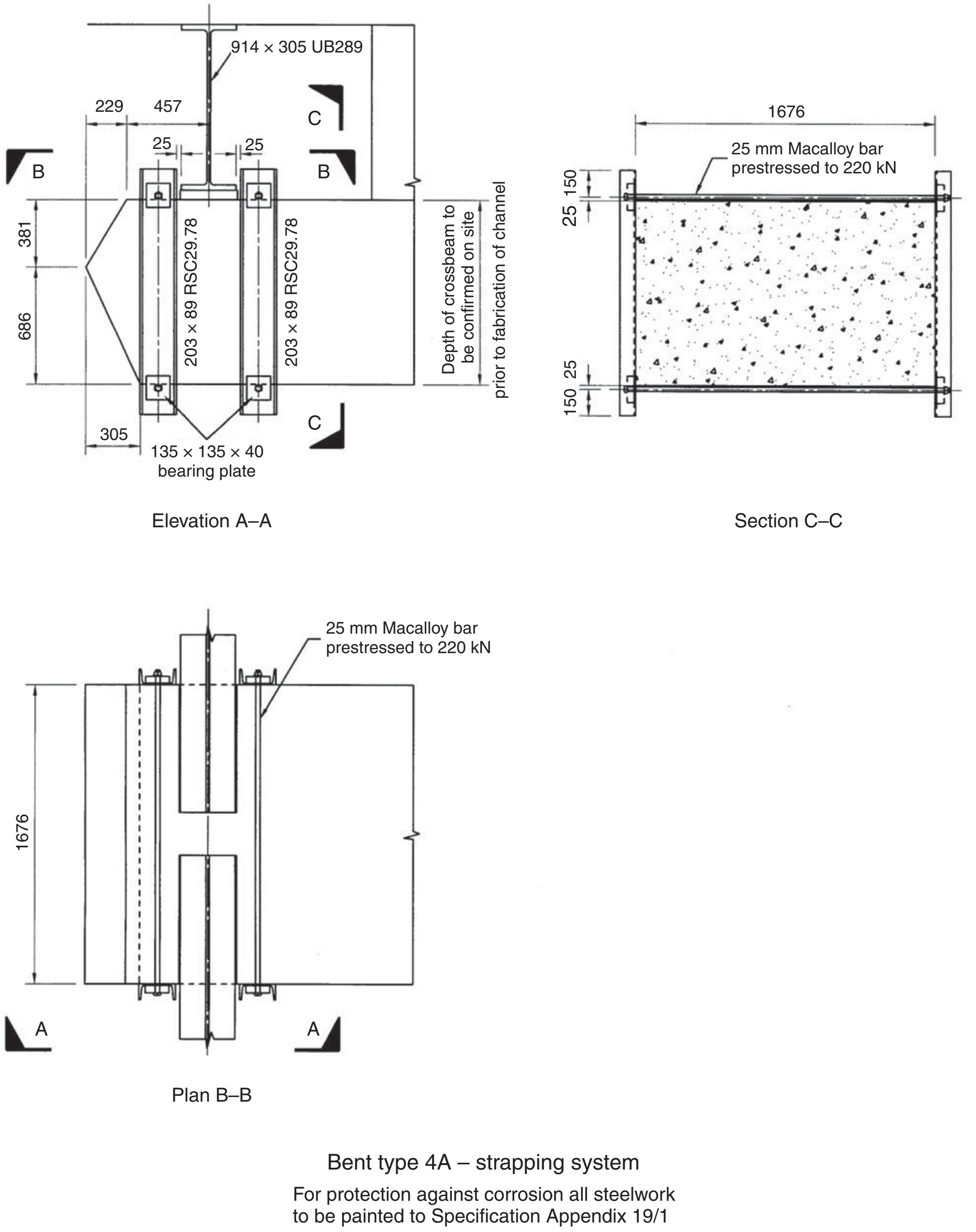

Figure 20.2 – Temporary strapping for beam end cracking

Following strapping of the beams the cracks were grouted with epoxy resin – the vertical cracks under pressure to achieve maximum penetration where the straps would restrain the pressure – and the horizontal cracks under vacuum to achieve an adequate seal without opening the cracks further. The diagonal cracks under the bearings away from the beam ends were also pressure grouted as the stirrups in that region could prevent excessive opening of the cracks.

After filling the cracks, the diagonal cracks were cored to determine their depth, the grouting avoiding the cores breaking up on removal from the beam.

The greatest depths of diagonal cracks were found to be 80 mm and 125 mm, sufficiently shallow to consider that the integrity of the imaginary diagonal concrete strut supporting the outer bearings was not significantly impaired.

Resurfacing of the carriageway then required hardshoulder running, and a monitoring regime was set up to ensure that further deterioration did not occur.

Beam ends were recommended for waterproofing with silane following crack filling to encourage the beam ends to dry out and limit the development of ASR.

Repairs

The main weakness of the crossbeam ends was the absence of a strong reinforcement cage, there being only three small diameter reinforcement bars across the beam end. The permanent solution was therefore to repair the beam ends in the normal manner, replacing the concrete skin with flowable repair concrete to behind the reinforcement, and while doing so, to insert additional transverse reinforcement around the full perimeter of the beam (see Figure 20.3).

The amount of additional reinforcement required to overcome the deck contraction forces was calculated as four 20 mm diameter high yield bars for beams carrying two 50 foot spans and six 20 mm diameter high yield bars for beams carrying 70 foot and 90 foot spans within the end 1.3 m length of the beam. As the minimum bend radius for high yield bars is greater than for mild steel bars, the additional bars would occupy a slightly greater width of beam, and the crossbeams were widened marginally towards the ends to cater for that. (There was not room to accommodate sufficient mild steel bars.)

Conclusion

While the relative predominance of shrinkage, deck contraction and ASR could not be determined in relation to the overall problem, the permanent solution of additional reinforcement was beneficial in relation to all three effects. With the beam end cracking becoming widespread it was recommended that all crossbeams being repaired had additional reinforcement cast into the beam ends. It was also recommended that bearing frictions be measured and consideration be given to greasing the bearings if these were found to be continuing to increase. Further monitoring of crack widths and of ASR development was also advised.

Conclusion

The case studies described in the book provide an insight to techniques developed for the diagnosis, monitoring, management, temporary bracing and permanent repairs and strengthening of a range of problems encountered in various concrete bridge elements.

A few nuggets came to light during the design and construction which are possibly worth summarising.

When encountering extensive lengths of viaduct contaminated with chlorides, it was found worth repairing some of the less badly delaminated areas first, as by catching them in time before they had to be propped for repair, much money could be saved.

The stiffness of propping needs to be considered in addition to its strength.

There appeared to be significant advantages in element replacement rather than repair, on the basis that it might be a more straightforward and quicker procedure, and more reliable in the long term. However many complications developed with the need to maintain the overall articulation of the structure during the period the element was removed and these led to drawn-out theoretical and practical provisions and subsequently significant additional costs. The engineer needs to consider these implications in considerable detail before deciding upon whether repair or replacement is the preferable option. In some cases, however, replacement is the only viable option.

The shrinkage and creep of replacement elements need to be taken into account for their effects on the temporary and permanent structures.

When a structure is found to be defective, although there may be a need to prop it as a matter of urgency, it is nevertheless important to consider whether the emergency propping will provide sufficient strength and enable access for the permanent repair or strengthening. Once the emergency propping is in place it will be very difficult to remove it, so it is essential it is also suited for the refurbishment phase.

Bridge repair and strengthening can be a much more stimulating cognitive process than a green field site design since everything must be fitted into and around the structure that is already there. Modifying or propping a structure can have many unintended consequences and it is a perpetual and necessary challenge to work out in advance what these may be.

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